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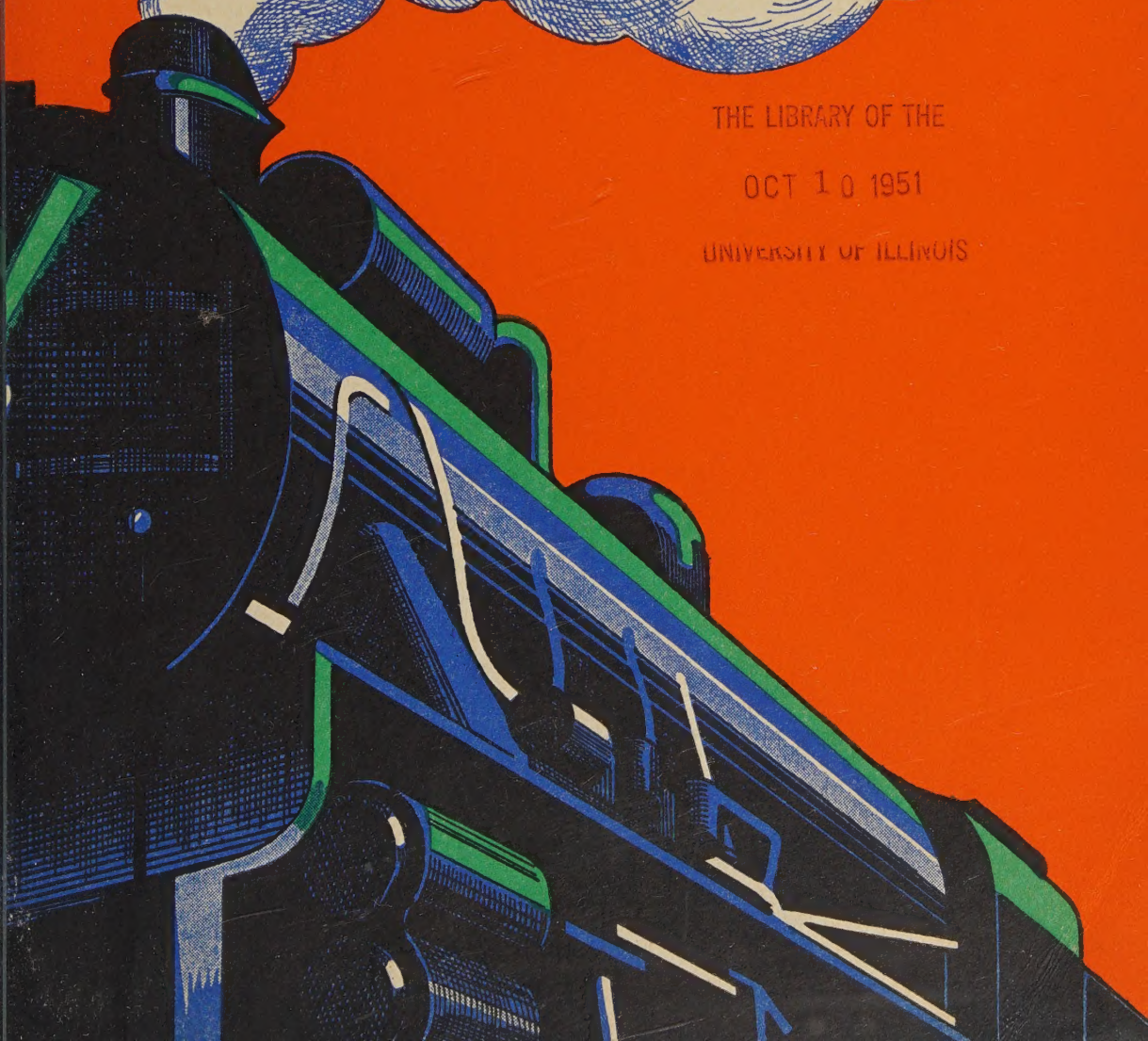
(English Edition)



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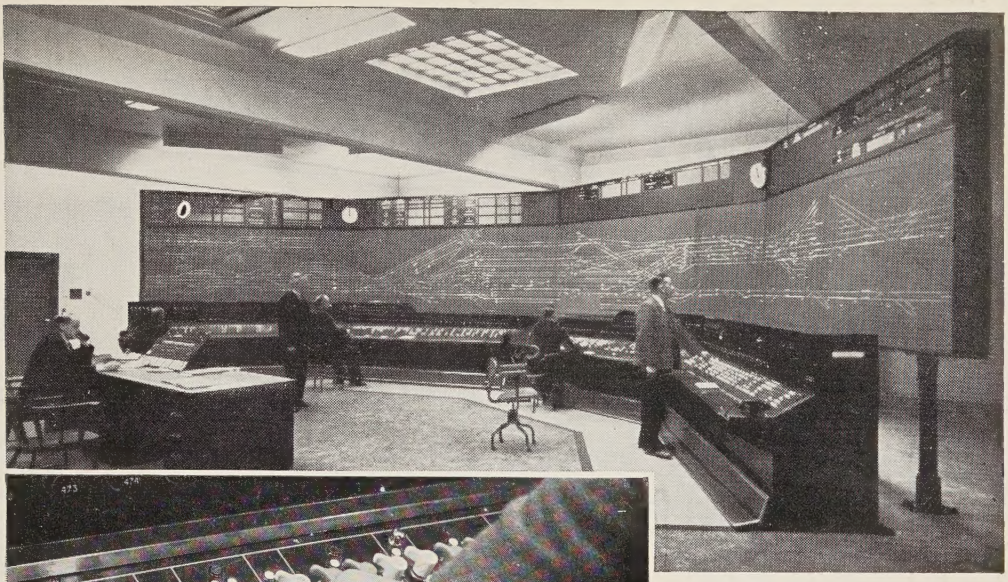

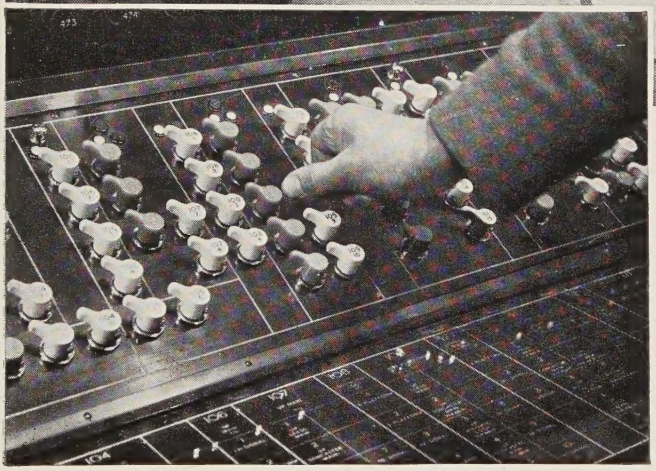

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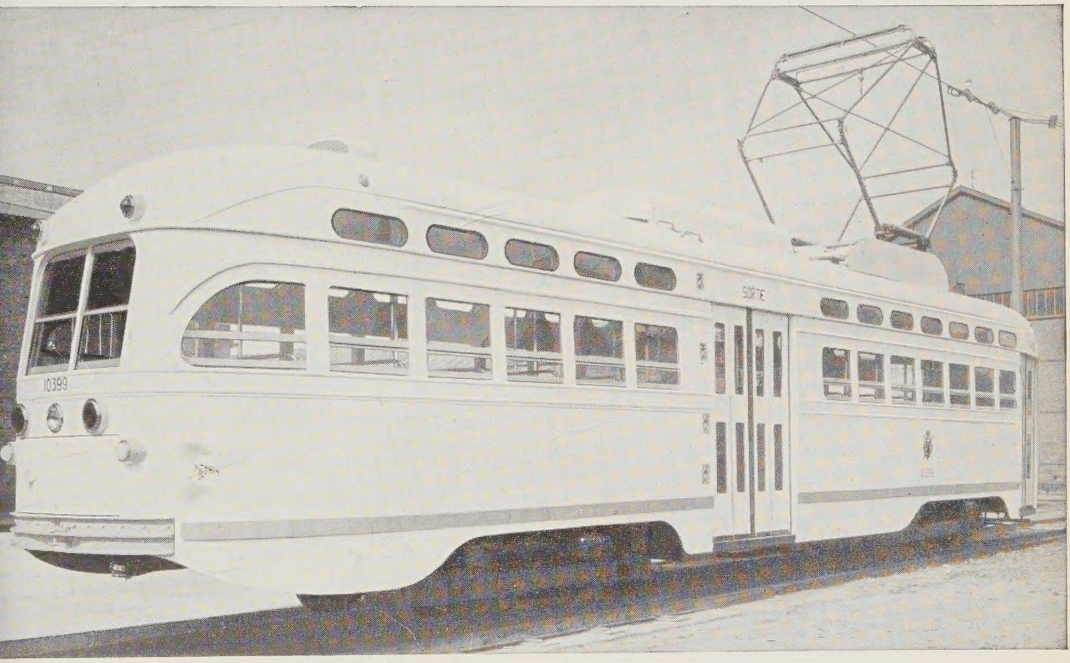

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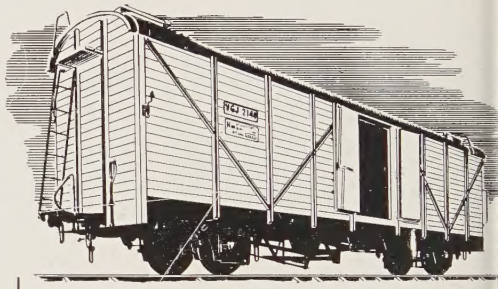

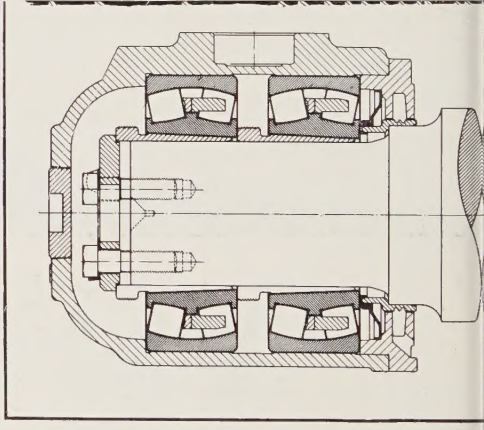

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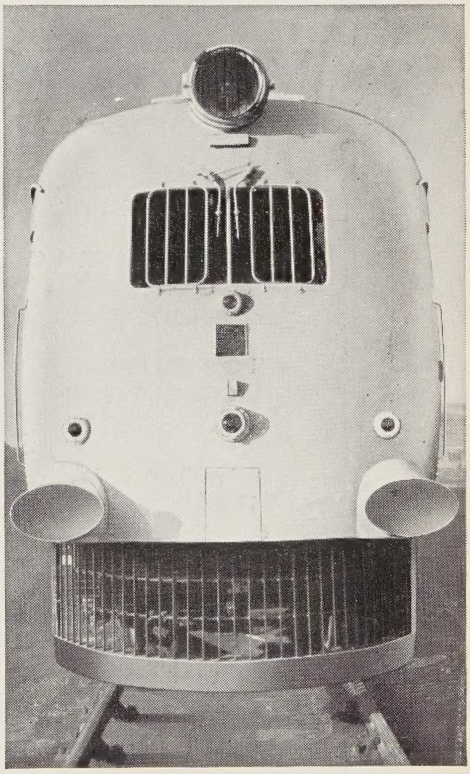

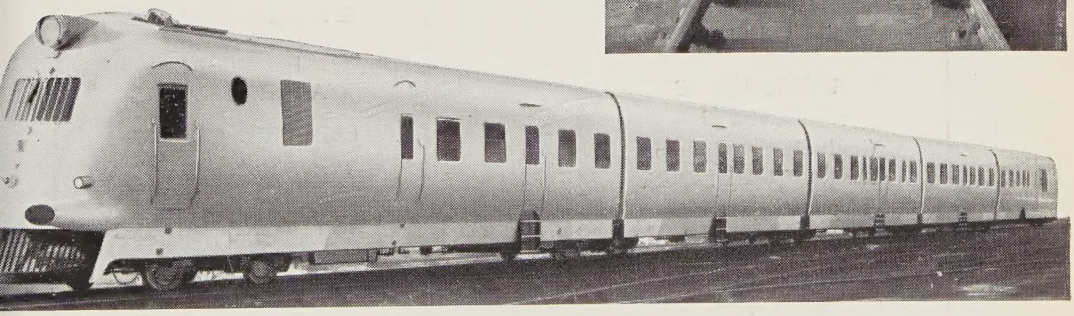

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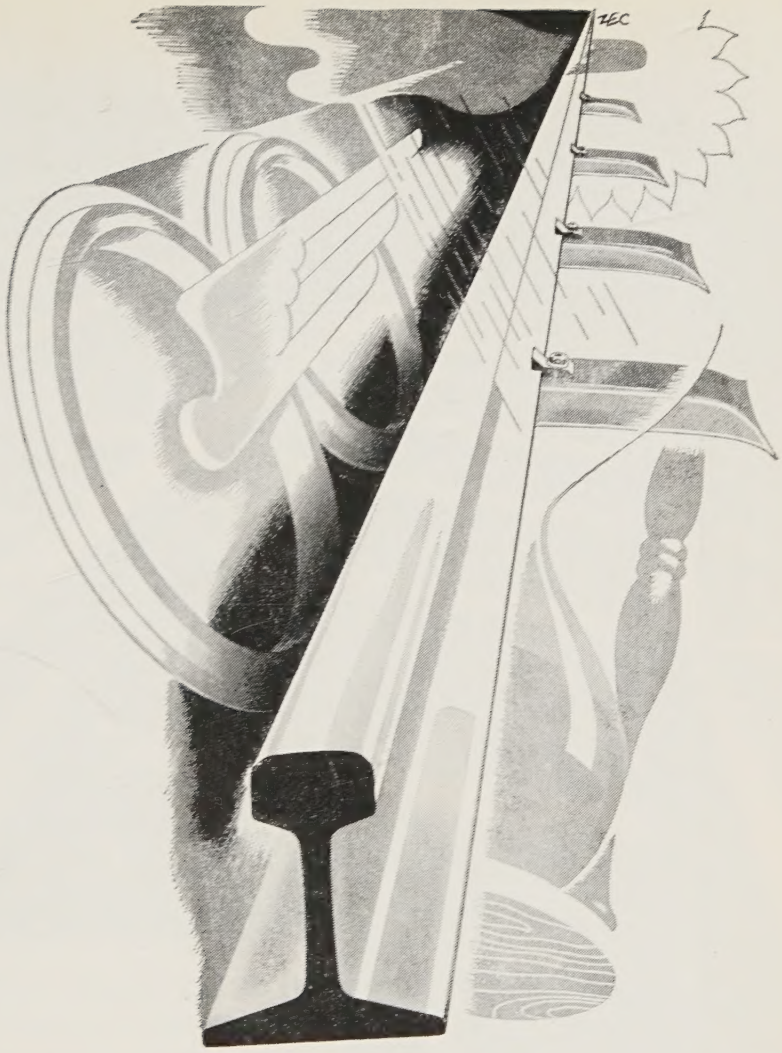

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OF THE

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CONTENTS OF THE NUMBER FOR SEPTEMBER 1951.

	CONTENTS.	Page.
I.	General consideration of **results of tests made on the materials obtained from iron and steel bridges** blown up by dynamite in 1940, by J. SERVAIS	575
I.	Calculation of the dimensions of the **masonry lining of tunnels,** by LUIGI LAMAGNA	599
I.	**Freight cars.** Repair, rebuild or buy new? by WILLIAM WYER.	635
V.	**Obituary :** LÉON JACOBS	645
V.	MONTHLY BIBLIOGRAPHY OF RAILWAYS	65

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An edition in French is also published.

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BULLETIN
OF THE
INTERNATIONAL RAILWAY CONGRESS
ASSOCIATION
(ENGLISH EDITION)

[62 (01 & 669]

General consideration of results of tests made on the materials obtained from iron and steel bridges blown up by dynamite in 1940,[1]

by J. Servais,
Inspecteur technique principal à la Société Nationale des Chemins de fer Belges.
(From the *Bulletin de la Société Royale Belge des Ingénieurs et des Industriels.*)

Let us first remember that the Belgian National Railway network covers altogether 2319 metal superstructures, as follows :

408 superstructures forming 365 road bridges and 1 911 forming 888 railway bridges.

As will be seen, in addition to bridges formed by single or double track superstructures, many works consist of several bays, either in the form of multiple cross girders or as a central cross girder with platform approaches, or again of platforms connected in pairs side by side; according to the requirements which may call for various combinations.

In May 1940, 60 overbridges and 295 rail bridges were destroyed to delay enemy advance and these systematic demolitions affected the whole of the network (fig. 1).

These figures, which include the greater portion of the decked metal bridges (tabliers) given in per cent in relation with the total number of structures, would not give a satisfactory indication of the extent of the destructions effected owing to the fact that works destroyed were usually those of major importance.

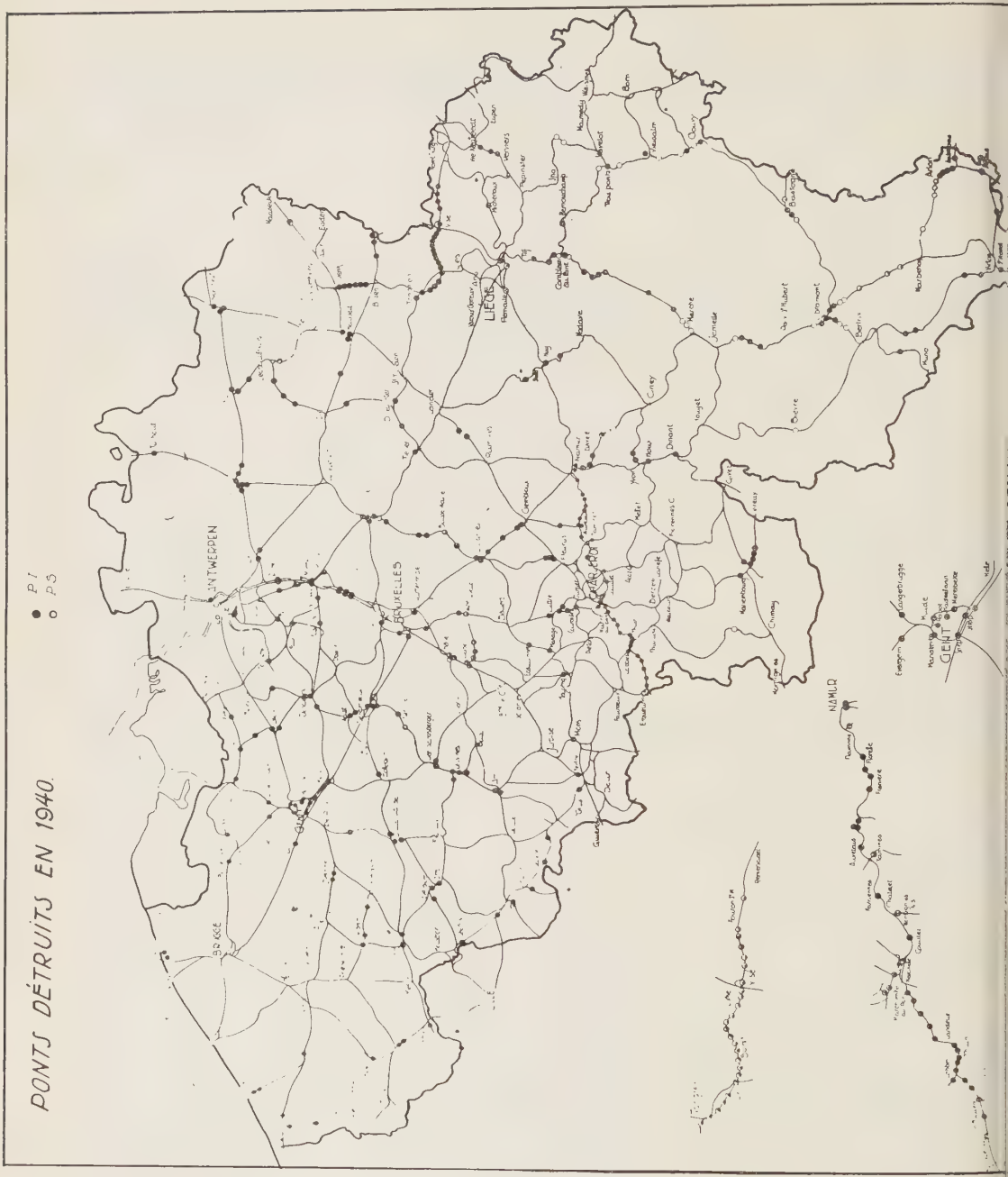
One can form an idea of the importance of these demolitions when it is known that at the end of 1941, after it had been decided for reasons of economy and in order to save time, to use up such parts — to the fullest possible extent — as are in condition for re-use, the weight of steel required for reconstruction and reconditioning of the superstructures, amounting to some 54 000 tons. This weight includes the reinforcement of the concrete structures.

At the beginning of 1944, the majority of works for important lines had been rebuilt and put in service.

Afterwards, fresh serious damage was recorded; some due to sabotage by the « Resistance » in order to delay the enemy; and others by withdrawal operations which occurred during the liberation of the country.

The tests referred to in the present memorandum concern the destructions of May 1940, these being moreover the most important ones. The trials only concerned

(1) Summary of the lecture given in Brussels on the 3rd January 1945, at the « Société Royale Belge des Ingénieurs et des Industriels ».


PONTS DÉTRUITS EN 1940.
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Bulletin of the International Railway Congress Association

CONTENTS OF THE NUMBER FOR SEPTEMBER 1951.

1951 **62** (01 & 669

Bull. of the Int. Ry. Congr. Ass., No. 9, Sept., p. 575.

SERVAIS (J.). — General consideration of **results of tests made on the materials obtained from iron and steel bridges** blown up by dynamite in 1940. (9 000 words, tables & fig.)

1951 **625** .13

Bull. of the Int. Ry. Congr. Ass., No. 9, Sept., p. 599.

LAMAGNA (L.). — Calculation of the dimensions of the **masonry lining of tunnels**. (6 000 words, tables & fig.)

1951 **625** .24 (73)

Bull. of the Int. Ry. Congr. Ass., No. 9, Sept., p. 635.

WYER (W.). — **Freight cars.** Repair, rebuild or buy new? (5 000 words, tables & fig.)

1951 **385** (09.2

Bull. of the Int. Ry. Congr. Ass., No. 9, Sept., p. 645.

OBITUARY NOTICE. — Léon JACOBS. (500 words.)

riveted bridges, the welded ones being in any case very limited in number.

Among the principal superstructures destroyed, were those on the Albert Canal, on the Meuse, the Sambre, the Escaut (Scheldt) and also the big « Vierendeel » bridge across the canal at Malines.

In their state of demolition and after the exceptionally severe stresses due to the explosions and to the impacts when parts of the spans were broken up and fell down between the abutments, the structures subjected to dynamite explosions became the object of examination on the site.

The observations made a favourable impression on the examiners and broadly speaking it appeared that the destruction by explosive charges were restricted to those parts which had been actually torn apart without further serious effects on the parts that remained intact.

However, less superficial control was called for and in the beginning of 1941, we were instructed to examine, by soundings, the material of some of the structures, with the object of ascertaining whether the rough treatment they had suffered might not have caused following sudden and abnormal stresses in the parts blown up or in neighbouring parts, metallographic modifications which might affect the character of the materials forming the beam or other parts and thereby endangering the safety of the structure, in case of re-use.

At this point, it was agreed that the tests should affect :

1° an *iron* structure, the single track Thames bridge, built in 1868; of which the cantilevers were now lying at the bottom of the Scheldt;

2° a *steel* bridge, the large Vierendeel, with double tracks, having a nominal span 63 m. 140 (207 ft.), across the canal at Malines, built in 1933 and which had been in use for 7 years at the time it was destroyed.

It was the tests carried out on these two structures, which originated the experimental tests undertaken afterwards on numerous superstructures and for this reason we shall first of all give the data provided by the tests.

I. — THAMES IRON BRIDGE.

The specimens were obtained from an approach superstructure of 40 m (131′ 2 3/4″) nominal span, which had been taken out of the water.

The tests were made on a plate of 500 × 10 mm (1′ 11/16″ × 4″) taken from a piece of the upper part of one of the straight trellis girders situated close to the explosion.

The series of tests which we were able to make with the means at our disposal included :

a) a macrograph taken of a print by the Baumann method;

b) the chemical analysis;

c) tensile, bending and resilience tests;

d) micrographic examination;

e) in certain cases, durability tests under repeated impacts.

The results were as follows :

A. — MACROGRAPHIC EXAMINATION. — The Baumann prints failed to show any serious segregation of sulphur compounds, but they showed numerous inclusions, visible to the naked eye, on the polished part and forming as many interruptions in the continuity of the solid body.

B. — CHEMICAL ANALYSIS. — The drillings were taken by milling over the whole transverse section of the plate (semelle) :

S = 0.051 %; P = 0.52 %; C = 0.03 %; Si = 0.09 %; Mn = 0.04 %.

These results are the mean analysis of the different chemical elements; the very high phosphorus content will be noted. This figure of 0,52 % represents about 10 times the quantity normally met with in steels of good average quality (Table 1).

TABLE I.

Description and position	Elastic limit kg/mm²	Breaking stress kg/mm²	Elongation %	Contraction %
I. Close to fracture from explosion : lengthwise.	30.8	35.2	3.5	nil
II. In zone at a distance from the fracture :				
lengthwise.	30.8	33.0	2.8	nil
crosswise	24.4	30.0	1.4	nil

C. — TENSILE TESTS. — The flat test pieces are of the type sanctioned by our Company and respond to the formulae :

$$L = \sqrt{66.67\ S}$$

L being the distance between marks on the test piece and S its initial section in mm².

We would observe that the elastic limits are relatively high compared with the breaking stress. The elongations are very slight and the contractions nil.

The fractures are lamellar and show a dull surface with a few bright spots. No difference is to be seen between the results of tests on material in the vicinity of the explosion and from tests pieces taken from the zone most distant from the explosion and not having been affected by the latter.

D. —BENDING TESTS. — The test bars are 40 mm wide × 250 mm long (1 9/16″ × 9 27/32″).

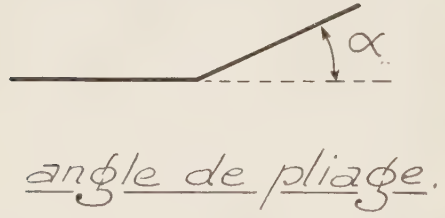

Fig. 2. — Bending angle.

One test bar cut lengthwise and one transversely were taken out just beyond the point of fracture caused by the explosion. In the lengthwise test bar, partial fracture of the part subjected to tension, took place at 30° angle of bend. In the transverse test bar complete fracture took place at 25°.

The fractures are dull with a few bright spots.

Hence the ductility of this iron is low.

E. — RESILIENCE TEST. — Test bars of the Mesnager type with a slot 2 mm (25/32″) deep rounded at the bottom and cut at right angles to the main rolled faces.

a) vicinity of the fracture caused by the explosion :

lengthwise : mean of 4 tests : 2.18 kgm/cm²

Fig. 3.

Thames Bridge — 1868

Approach superstructure : nominal span 40 m (131′ 2 3/4″).

Slag inclusions.

Analysis : S = 0.051 %; P = 0.52 %; C = 0.03 %; Si = 0.09 %; Mn = 0.04 %.

75 ×

Fig. 4.

Boom Bridge — 1875.

Nom. span : 61 m (200′ 1 5/8″).

Slag inclusions found according to the direction of rolling. — Analysis : S = 0.031 %; P = 0 49 %; C=0.014 %; Si = 0.10 %; Mn = trace.

75 ×

Fig. 5.

Remouchamps Bridge — 1882.

Nom. span : 60 m (196′ 10 1/4″).

Slag inclusions. Reticular structure.

Analysis : S = 0.018 %; P = 0.2 %; C = 0.06 %; Si = 0.08 %; Mn = 0.19 %.

b) zone distant from the fracture :

lengthwise : mean of 4 tests : 2.15 kgm/cm².

The means indicated above were obtained from readings only slightly dispersed. They characterize a metal which is fragile to shock in bars which are bevelled off.

There is no marked increase of fragility in the zone close to the fracture due to explosion as compared with the more distant zone.

F. — MICROGRAPHIC EXAMINATION. — Specimens taken from different zones indicate the frequent and considerable slag inclusions distributed in the body of metal and placed according to the direction of rolling.

The micrography (Fig. 3) shows an average picture of the extent of the slag inclusions in the iron of the Thames Bridge. Fig. 4 & 5 refer to two other structures and similarly give an indication of the distribution of more or less considerable inclusions which may be found in the iron.

The results of the above tests are concordant and lead inevitably to the conclusion that this quality of iron is inferior; phosphorus content excessive, elongations and contractions very low, ductility very poor, resilience shows great tendency to fracture under shock and finally the existence of numerous slag inclusions of varying importance shown by micrographic examination.

The evidence is particularly bad if we judge the results by comparison with present requirements in mild steels for structural work.

As a matter of fact, the general specification for standard steels for bridges, iron structures and water towers, lays down :

a) for all rolled sections measured in the direction of rolling :

elastic limit E : 24 kg/mm; (15.24 tons per sq. inch) minimum;

ultimate breaking strength R : 38 to 44 kg/mm; (24.13 to 27.94 tons per sq. inch) minimum;

elongation A = 25 %.

b) for sheets : in addition to the above conditions a transverse elongation A of 20 % min.

Moreover, the test bars selected from the materials must be capable of resisting bending of parallel strips separated by half their thickness, without showing any cracks or trace of disintegration.

On the other hand, these directions compare noticeably with those which at that period, defined the range of qualities of irons used.

This range which we extract from the 1902 records at the Cockerill works, stood as follows :

TABLE II.

Breaking stress kg/mm²	Elongation %	Quality No.
32	6	2
33	10	3
35	15	4
37	18	5
38	20	fine grained

The specifications in use at that time call for No. 3 iron as the particular material to be used for metal structures; the No. 4 iron being reserved for rivets and bolts.

Among the conditions of manufacture, we find that the characteristics of the iron were varied according to their particular purpose, whether it was a question of sheets, bars and sections or for wide plates.

As a result of experiments made with the Thames Bridge upper flange plates, it was laid down, in October 1941, that all the old iron superstructures must be subjected to tests of the material so as to establish a classification, if such should be required, stating the order of priority according to which renewals of these old parts should be governed.

It was understood that we should in the first place examine the superstructure demolished in May 1940 and reconstructed later.

It should be understood *a priori*, that the puddled iron is only of retrospective interest since the iron has long ago been replaced by mild steel in our structures; but it will no doubt be realized how astonishing is the number of iron superstructures still in service on our network and on the main lines.

The material used on 82 superstructures was sounded in the same way as was done in the case of the Thames Bridge.

Fortunately, all these structures which in general had well resisted oxidation, did not give such poor results as was the case with the Thames bridge, and a good number of them gave such good results as to allow them to be classified as « good » irons, No. 4 and 5 in the range referred to above.

The overall variations in the characteristics (lengthwise) of the whole of the superstructures in iron which were examined, were as follows :

The elastic limits varied between 19 and 35 kg/mm² (12.06 and 22.22 tons per sq. inch); the breaking strengths from 25 to 43 kg/mm² (15.87 to 27.30 tons per sq. inch); the expansions from 1 to 26 %.

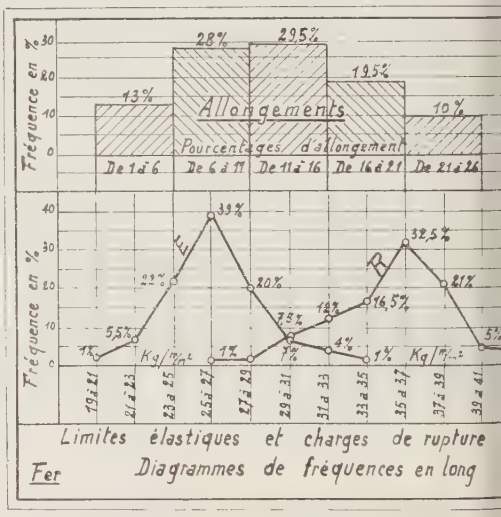

Fig. 6. — For iron.

Explanation of French terms :

Allongements = Elongations. — Fréquence en % = Frequency in %. — Fer. Limites élastiques et charges de rupture. Diagrammes et fréquences en long. = Iron. Elastic limits and breaking loads. Lengthwise frequency graphs.

The following frequency graphs give the « percentages reached » for the whole of the tests made (Fig. 6).

These considerable differences in the results may at first surprise, in particular as regards to the elongations, but actually they are not extraordinary, as Couche in his book (1873 edition) on the railways states that elongations for irons may vary between 1.7 and 17 %.

Measured transversely, 67 % of the expansions are less than 2 % and the remaining 33 % lie between 2 and 5 %.

These results compared with the conditions laid down for steels, easily tend to judging the majority of the materials examined as being unsatisfactory, especially if we take into consideration the lack of ductility shown by the bending tests.

Hence, we found ourselves obliged to undertake a more detailed analysis of the phenomena characterising the behaviour of the iron on the one hand and of the steel on the other.

We could not in fact without further examination, call materials poor which stood up for more than 60 years in our structures without failure taking place in service.

This is why, we took advantage of the opportunity to follow up our experimental investigations on the iron used in numerous structures erected at different periods, and we undertook a more far reaching analysis of the factors which would enable us to arrive at a sound and more definite estimate of the actual value of puddled iron which was the material used for our former metal structures.

Let us therefore take up point by point, the data obtained from the flange plates of the Thames bridge, at the same time attaching the comments called for by the examination of other bridges.

A. — CHEMICAL ANALYSIS. — What particularly kept our attention from the start, was the very high phosphorus content of the irons we examined; we should remember that the plates of the Thames bridge showed a content of 0.52 %.

Of the 82 bridges examined, 135 complete analyses were made, of which we will only consider the phosphorus, since the quantities of other measured chemical elements could be considered as normal.

Fig. 7 shows the phosphorus content and its periodic variation.

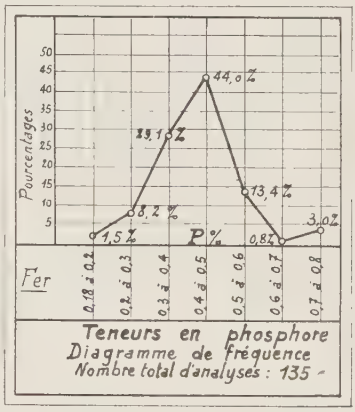

Fig. 7.

Explanation of French terms :

Pourcentages = Percentages. — Fer = Iron. — Teneurs en phosphore = Phosphorus contents. — Diagramme de fréquence = Frequency diagrams. — Nombre tota d'analyses : 135 = Total number of analyses : 135.

61.2 % of the specimens showed a phosphorus content reaching or exceeding 0.4 % and going up to 0.8 %.

Only 1.5 % of the phosphorus contents were lower than 0.2 %.

We were not unaware of the fact that when analysing puddled iron, the phosphorus content is always very high, owing to the fact that the cuttings contain, unavoidably, slag inclusions, where phosphorus is much more abundant than in the surrounding iron.

It is estimated that about 1/4 to 1/5 of the total phosphorus is contained in the slags included in the puddled iron and it is generally admitted that the remaining phosphorus gives rise to the formation of Fe_3P, which enters into solid solution with the iron.

Hence, this iron consists of polyhedral

grains of ferrite containing the Fe_3P in solution.

It is simple to calculate the amount of Fe_3P formed. In effect the atomic weight of the iron being 56, and that of the phosphorus 31, we get for the phosphorus :

$(3 \times 56) + 31 = 199$ particles of Fe_3P.

One part of P therefore gives rise to approximately 6 parts of Fe_3P.

If we return to our total content of 0.52 % of the Thames bridge, we shall have : 1/5 or say 0.10 % retained by the slag and about 0.4 % available for forming Fe_3P, which gives us $0.4 \times 6 = 2.4$ % of Fe_3P in solution in the iron, hence the increase in fragility.

Actually, and to a certain degree, the iron thus contains less P than would appear from the analysis; but this point being admitted, it was not any the less true that the content of 0.52 % appeared to us to be excessive.

In order to get confirmation of our impression, we asked for the opinion of the Management of the « Forges et Laminoirs de Baume » at Haine-Saint-Pierre, the only Belgian works still making use of puddled iron.

Their reply was as follows :

« The French specification places at 0.14 % the maximum content in phosphorus for puddled iron used in the production of boiler tubes for the Mercantile Marine.

« The same maximum content in P is specified for puddled bars used for this purpose.

« The puddled iron produced by Baume, seldom contains more than 0.10 % P.

« We estimate » continues this letter, « that the P content must not exceed 0.10 % for posts subject to motor shocks.

« Nevertheless, we are of the opinion, that phosphorus contents of 0.10 % to 0.20 % may very well be admitted in ordinary structural works ».

Thus our view that iron containing sensibly more than 0.20 % of phosphorus should be considered with suspicion, is confirmed.

On the other hand, in his « Manuel de la Métallurgie du fer », 1895 edition, the German Professor LEDEBUR in discussing phosphorus expresses himself as follows (Vol. II, p. 214) :

« Wrought iron often contains 0.3 and even 0.4 % of phosphorus; under these conditions it is certainly fragile, but it can nevertheless be used for a certain number of current uses. »

The same author also says (Vol. I, p. 322) :

« The influence of this element (phosphorus) on the mechanical properties of iron is very marked; a small quantity does not appear to affect the tensile strength where there is no shock action, a slightly increased amount however reduces it. »

« The elastic limit and the coefficient of elasticity are higher, ductility is reduced; increased brittleness is noted; vibrations which are sometimes very slight, lead to fracture in iron containing phosphorus; such fractures often take place, once the elastic limit has been passed. »

One cannot give a better reply to the question raised.

We may now pass to the tensile tests.

These results of tensile tests have likewise called our attention and were worth of a detailed analysis.

We give diagrams of elongation under load of iron and of mild steel under normal average conditions in Fig. 8.

We observe that the elastic limits are of the same order; on the other hand, the zone of plastic deformation. very restricted in the case of iron is very considerable for mild steel. This zone includes a period of elongated contraction for the steel, whereas in the case of iron this period is usually very short, at times almost equal to zero.

Exception may be made however for some structures in which the specimens tested, indicated a zone of plastic deformation approaching that of steel (particularly the test bars taken from the Remouchamps bridge, built in 1882.)

The diagrams also show tests in the transverse direction and these indicate an even

more marked difference as regards plasticity.

From an examination of these diagrams we gather that the iron, generally speaking presents a satisfactory elastic limit approaching that of mild steel. Once this limit is exceeded, the plastic deformation becomes very restricted and in many cases leads to a more or less rapid disintegration of the material.

It is even permissible to assume that the disintegration starts at certain points before the elastic limit has been reached.

If we consider that the slag inclusions tend to break the continuity of the mass of iron which envelopes them, il will be readily appreciated that the greater the number and size of slag inclusions, the more the inherent capacity for deformation of the iron will be reduced, and the more easily disintegration will set in.

This effect becomes clear from mechan-

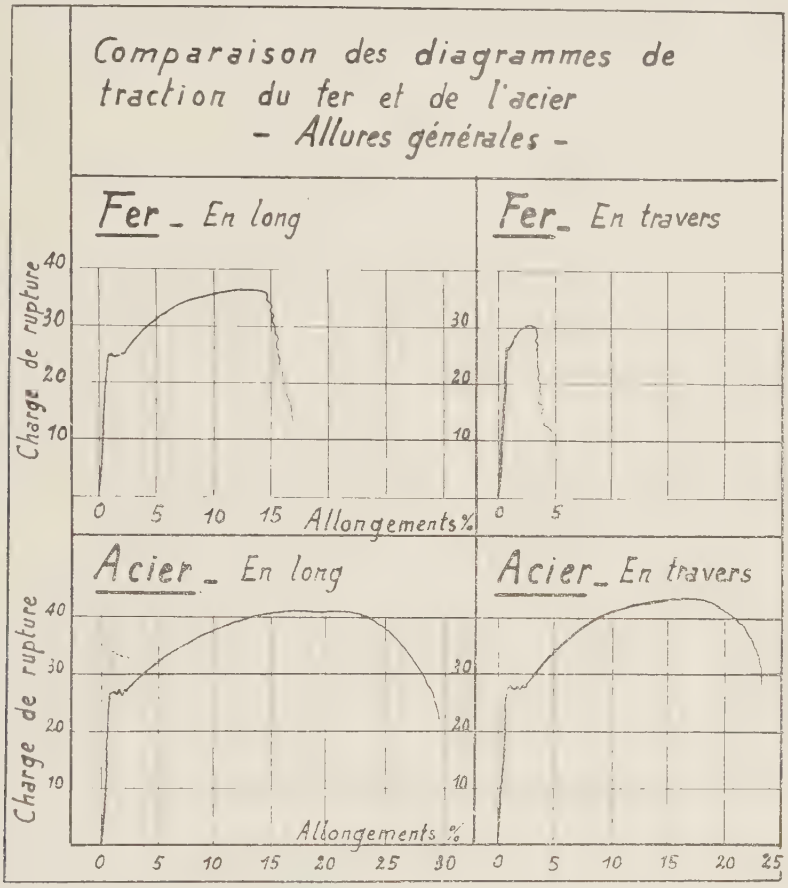
Comparaison des diagrammes de traction du fer et de l'acier
- Allures générales -
Fer - En long
Fer - En travers
Acier - En long
Acier - En travers
Charge de rupture
Allongements %
0 5 10 15 20 25 30
10 20 30 40

Fig. 8.

Explanation of French terms :

Comparaison des diagrammes de traction du fer et de l'acier. — Allures générales = Comparison of the diagrams of elongation of iron and steel. — General views. — Charge de rupture = Breaking load. — Fer-En long = Iron. — Lengthwise. — Acier = Steel.— En travers = Crosswise. — Allongements = Elongations.

ical considerations. These inclusions act as slots, which tend to favour a fracture due to the high stresses set up locally.

In fact, if we go back to the initial treatment of the iron, it can be said that the quality of the puddle bars forming the blooms, their degree of refinement and their temperature at the moment of refinement, govern the final quality of the iron. The presence of important slag inclusions is a sure indication of insufficient refinement.

This being so, we shall not be surprised to find in one and the same bar, strengths, elongations and contractions which vary considerably. We give below a few instances :

TABLE III.

1. — *Tests made on two plates from the underbridge from the Mandeville Road to Liège-Guillemins.*

1st plate :

Sample No 1, lengthwise	E	= 22.75 kg/mm² (14.44 tons per sq. inch)
	R	= 31.6 kg/mm² (20.06 tons per sq. inch)
	A %	= 13.6.
Sample No. 2, lengthwise	E	= 23.15 kg/mm² (15.55 tons per sq. inch)
	R	= 29.45 kg/mm² (18.69 tons per sq. inch)
	A %	= 5.7.

2nd plate :

Sample No 1, lengthwise	E	= 23.7 kg/mm² (15.04 tons per sq. inch)
	R	= 29.45 kg/mm² (18.69 tons per sq. inch)
	A %	= 5.0.
Sample No. 2, lengthwise	E	= 23.67 kg/mm² (15.02 tons per sq. inch)
	R	= 32.8 kg/mm² (20.82 tons per sq. inch)
	A %	= 11.4.
Sample No. 3, crosswise	E	= could not be ascertained
	R	= 21.45 kg/mm² (13.61 tons per sq. inch)
	A %	= 2.0.

2. — *Tests made on materials taken from the bridge at B. K. No.* 14486 *of the Lembecq to Ecaussines lines.*

Sample No. 1, lengthwise	E	= 33.00 kg/mm² (20.95 tons per sq. inch)
	R	= 41.8 kg/mm² (26.53 tons per sq. inch)
	A %	= 6.66.
Sample No. 2, lengthwise	E	= 32.40 kg/mm² (19.93 tons per sq. inch)
	R	= 35.6 kg/mm² (22.60 tons per sq. inch)
	A %	= 2.96.

We could quote plenty of other instances equally striking.

It should be added that when taking and testing several pieces from the same superstructure, we obtain varying results, especially regarding elongations and contractions.

The quality of ductility, closely related to plasticity which we referred to, is likewise seen in the bending tests.

Numerous test bars, in longitudinal section, fracture in part at the zone stressed at an angle less than 60° and when the test pieces resist bending stresses in parallel sections, the analysis often indicates a phosphorus content below 0.4 % and low slag inclusions.

Cut in cross section, the majority of the test pieces show high fragility and are broken by very low stress, sometimes even below the elastic limit observed in longitudinal sections.

We will now see how the iron resists the dynamic stresses.

Resilience being defined by the work done on a test bar slotted to a predeterm-

ned section, stated in kg/cm;, it will be ppreciated that the shock stress to which ne bar is subjected in the course of the est will depend on the degree of plastic eformation in the neighbourhood of the ot.

The greater the zone of plasticity pre-ented by the metal under stress, the greater ill be its ability to deform under shock tress and hence the higher could be its esilience.

As we have seen, since the plasticity of ne iron under static stresses is restricted s regards the qualities which we are onsidering, we must not expect other than ow resiliences which point to rapid disint-gration.

All in all we have handled 91 sets of 4 est pieces in the longitudinal section and 2 sets of 4 test pieces in cross-section. The means recorded lie between 0.5 and kgm/cm² in the direction of rolling (long-udinal section) and between 0.5 and .75 kgm/cm² for the pieces cut in cross ection.

The few higher figures relate to specimen aken from the Remouchamps bridge and rom certain superstructures from the Je-nelle-Dinant line (B. K. 8 788).

The graphs showing its prevalence are iven below (Fig. 9).

In the longitudinal sections, the percent-ge of figures below 4 kgm/cm² reaches 5.7 %.

Of the cross-sections, no test piece gave kgm/cm² and 66.2 % showed figures less nan 1 kgm/cm². Thus, the predominant nfluence of phosphorus was noticeable in ne fragility of the specimen. A low content n phosphorus corresponds to a low content n slag inclusions, hence good refinement nd higher resilience.

When shock stresses are concerned, we ave seen that opinions are unanimous in ttributing to the presence of phosphorus n increase in fragility.

It remains to be stated that the micro-raphic examinations have given valuable information concerning the position, the distribution and the number of slag inclusions.

These inclusions are found in the form of strings and beads lying in the direction of rolling of the metal; this fact gives the fractures their characteristics appearance of laminations.

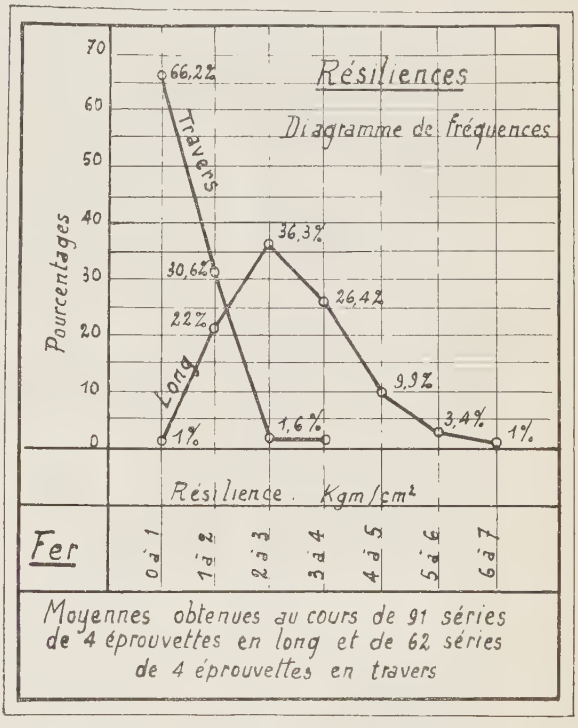

Fig. 9.

Explanation of French terms :

Résiliences = Resiliences. — Pourcentages = Percentages. — Diagramme de fréquences. = Diagram of frequencies. — Travers = Cross section. — Long = Longitudinal section. — Fer = Iron. — Moyennes obtenues au cours de 91 séries de 4 éprouvettes en long et de 62 séries de 4 éprouvettes en travers. = Averages observed with 91 sets of 4 test pieces in the longitudinal section and 62 sets of 4 test pieces in cross section.

We may summarize by stating that the fundamental property of the relatively high elastic limit of the iron explains its excellent behaviour in our structures.

Provided the working stresses are normal

and that we have only to deal with static stresses, the safety of the structure is assured. This condition is not maintained if shock stresses are to be feared and if the phosphorus content is high. In this case we must remember the fragility which is brought about by the relation between cause and effect.

We should not forged that the danger may be increased by reductions in cross-sections due to corrosion, since we had emphasized in passing the good behaviour of the iron in respect to corrosion, but it is certain that helped by the effect of weathering, the destructive effect of oxidation had taken place among the samples which were submitted to us, amongst which sections were found which presented considerable reductions in important sections.

A final observation may be made concerning our test results, namely that the data collected may perhaps very imperfectly reflect the properties of the metal parts when new and that these properties may have become modified by ageing?

We should remember that by « ageing » of iron and steel we understand the changes which take place in time at ordinary temperatures or at moderate temperatures, as a result of unbalanced conditions due may be to tempering being done below the a' point (about 720° C) or after cold forging: (tension, compression, torsion, cold drawing, etc.).

Then the metal tends to return by itself towards a more stable physico-chemical state of equilibrium, but this return is delayed by internal friction which constitutes passive resistances which are greater the lower the temperature at which the change has to take place.

The effects of slight supersaturation of carbon, oxygen and of nitrogen are shown by intergranular precipitation of microscopic or ultra-microscopic particles of cementite Fe_3C or by a precipitation of oxide Fe_3O_4; or of nitrite Fe_4N along the joints of the grains and along certain crystallographic planes. Under these forms, the precipitations result in a blocking of the lines of cleavage in the crystals and are the cause of increasing the resistance, the hardness and the elastic limit, while the elongation and above all the resilience diminish to a marked degree. Thus the metal has become more fragile. The

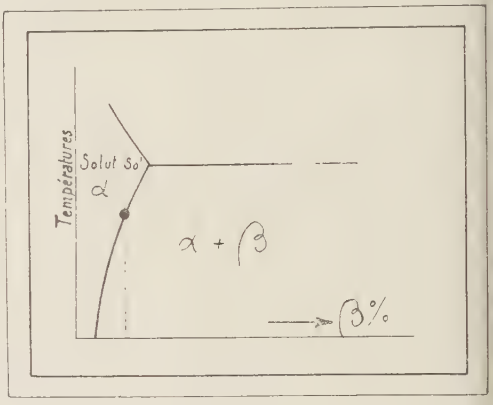

Fig. 10.

mechanism of these supersaturations arises from the graph of physico-chemical equilibrium which takes the same form in all three cases (Fig. 10). With the heating, the solubility of B in the solid solution increases as the temperature rises and when cooling takes place the solution deposits the hard constituants Fe_3C, Fe_3O_4 or Fe_4N.

The pieces of iron which we examined, did not disclose any signs of ageing during the mechanical and metallographic tests.

Reheating to 925° C of certain samples has not sensibly modified the mechanical characteristics as compared with the results obtained from the metal in its natural state. Moreover tests made on accessories such as adjusting clips gave results agreeing with those obtained from the other samples. It is therefore permissible to claim that the characteristics noted in the case of parts taken from the structures, are similar to those obtained from the metal in its original state or at most differ very slightly from those results.

At the same time, ageing cannot be

xcluded from the characteristics of the iron nd it is indeed certain that this metal is ubject to ageing in the sence which we ıave described, and if it does not take ɔlace to the same extent as in the case of teel at least it is subject to it within certain imits. Actually, the metal always contains . slight amount of carbon and nitrogen vhich may give rise to supersaturation ɔr cold working, to intergranular precipitaion of particles of cementite or of nitrides hat characterize the phenomena.

We may add, in order to complete our nvestigation, that we also made a certain ıumber of endurance tests by repeated hocks on cylindrical, non-slotted test pieces; ɔut these tests could only be made on amples exceeding 12 mm (15/32″) in hickness.

The values arrived at in the course of hese trials could only provide a useful nterpretation when we were able, further ɔn, to compare them with those found for he steels.

Hence forward however, we can confirm hat the figures obtained show, by their ariations the chemical, physical and nechanical diversity of the types of iron mployed.

In conclusion we would emphasize that he rivets used in the iron construction howed a higher quality of iron, much ɔurer as regards phosphorus content and nclusions.

The phosphorus content of all the rivets xamined was less than 0.2 %, which goes o prove that at the time when one used nediocre iron not very thoroughly worked nd having a high percentage of phosphorus, nanufacturers were fully aware of the need f using rivets and bolts of higher quality nd more thoroughly wrot, in order to emove the bulk of the slag inclusions. And s it is seen, they well knew how to proluce this.

Moreover, it would be unfair to our former ron masters, to imagine that no progress ad been made in the manufacture of iron. 'or example, it had not escaped their notice that the characteristics in cross-section were not a sufficient guide and gave rise to an actual risk. This is why, in order to correct this defect, the manufacturers had the treatment of the shearings forming the blooms before taking them to the forging press, rearranged so that in each faggot the bars were laid transversely and then staggered so as to obtain the best results after rolling. Improvements in the construction and working of the furnaces were likewise introduced from time to time.

We have investigated into the puddled iron or in any case the qualities of puddled used in the building of a good number of our superstructures.

We have tried to deduce from various tests the information which they have provided on the nature and on the properties of iron and also concerning its defects and in the final reports on each of our constructions we have not hesitated to point out the dangers of a material with properties so variable and so heterogeneous that it may be described as having a true tendency towards disintegration.

This is why we have proposed that apart from the conditions of stability, the contents in phosphorus found in the samples (specimens) should be considered as a factor to determine the urgency of renewals in iron superstructures, the state of oxidation and maintenance forming another factor which must be taken into account.

We cannot however close this chapter on iron bridges, without pointing out that at the present time iron of excellent quality is still being produced — of which the properties are well known and which are in no way inferior to the best qualities of mild steel.

We would specially note that iron always takes precedence over steel for ships chains and for haulage chains in collieries, on account of its excellent weldability, its good resistance to corrosion in sea water and its good wearing qualities.

We now can pass on to steel bridges.

II. — VIERENDEEL BRIDGE ACROSS THE CANAL AT MALINES — nominal span 63.140 m (207 ft.).

The part selected for test, weighed approximately 2 000 kg (1.96 t) and was taken from the upper parts of the superstructure.

This piece was projected by the explosion for a distance of about 100 m from its original position.

In the position originally occupied, this section of the superstructure consisted of 4 top flange plates forming the boom, each plate measuring 400 × 16 plus a portion of the bolt strap of the same thickness.

Before cutting, the piece in question was carefully examined.

It showed no cracks or defects, apart from the portions which were fractured, which could be attributed *a priori* to a lack of quality in the material.

The fractures at both sides of the piece, about 4.50 m in length, had not the same appearances.

On the one hand, the different elements; webs, angle irons and boom plates making up this fragment of the structure, showed clean fractures, with little or no deformation, while on the other they exhibited twisted parts, deformations and contractions of considerable importance.

The lack of deformation may be explained by the suddenness of the fracture due to the action of the explosive.

This type of fracture is what is known as « rupture de décohésion » (or in German « Trennbruch ») in which the explosive action causes a complete and rapid cessation of cohesion in a given plane.

In this case, there is no permanent deformation; hence the crystalline appearance of the intergranular fracture which sometimes led to a belief in a lack of ductility in the steel.

On the other side of the section, the plasticity of the metal was on the contrary able to play a part, in the sense that the crystalline area suffered slips at a certain number of levels, ultimately encountering each other and causing relative slip deformations until further movement became impossible, i. e. until the instant of fracture which was the final outcome.

In Germany, they use the term « Gleitbruch » for describing these plastic phenomena.

The tests were carried out on two boom plates, one exterior (top) and the other underneath the first, and also on three rivets, of these one rivet head was torn from its shank by the explosion.

The results are as follows :

A. — MACROGRAPHS. — The Baumann imprints show a central zone with slight elongation. No important concentration of sulphur was noted except a few points which were more marked, calling attention to the effervescence of the metal at the moment of its solidification.

B. — CHEMICAL ANALYSIS. — The cuttings were milled out over the whole transverse section of each boom plate :

TABLE IV.

Determination on	S %	P %	C %	Si %	Mn %
a) The outer plate . . .	0.048	0.049	0.07	trace	0.46
b) The inner plate . . .	0.068	0.068	0.09	trace	0.45

TABLE V.

Description and location of the tests	Limit of elasticity		Breaking stress		Elongation	Contraction of area
	kg/mm²	tons per sq. inch	kg/mm²	tons per sq. inch	%	%
1. Immediate vicinity of fracture without deformation :						
a) outer plate, lengthwise	25.5	16.18	39.0	24.76	21.9	62.3
b) inner plate, lengthwise	32.8	20.82	43.6	27.68	20.6	56.4
2. Continuous metal, unaffected :						
a) outer plate : lengthwise	24.2	15.36	37.2	23.61	31.3	64.0
crosswise.	24.0	15.24	37.0	23.49	26.2	45.9
b) inner plate : lengthwise	29.4	18.66	41.8	26.53	26.8	61.4
crosswise.	30.5	19.36	40.8	25.90	17.5	43.6

C. — TENSILE TESTS. — The flat test pieces agree with those accepted by our company and are defined by the formula

$$L = \sqrt{66.67\ S}$$

The above results show that in the zone close to the fracture without any plastic deformations, a certain amount of cold hardening is shown by an increase of the elastic limits and of the breaking stresses and by a reduction of the elongations. These reductions in the characteristics correspond to a state of ageing due to cold hardening.

Beyond this hardened zone, restricted to the vicinity of the fracture, the extent of which is not easy to determine, the normal properties of the metal reappear and the tests indicate that the characteristics are those of good quality mild steel.

D. — BENDING TESTS. — The test pieces are 40 mm (1 37/64″) wide and 250 mm (9 27/32″) long.

The two test pieces taken lengthwise from each of the plates, beyond the rupture zone, resisted bending through 180° (doubling) on itself without presenting fissures or cracks.

It was not possible to obtain bending test pieces in the neighbourhood of the fracture.

TABLE VI.

Description and location of the test pieces selected	Average resilience measured on sets of 4 test pieces
1. Zone adjacent to fracture without deformation :	
a) Outer plate, lengthwise	4.62 kgm/cm²
b) Inner plate, lengthwise	4.21 kgm/cm²
2. In zone of solid metal :	
a) Outer plate, lengthwise	10.99 kgm/cm²
b) Inner plate, lengthwise	10.25 kgm/cm²

E. — RESILIENCE TESTS. — The test pieces are of the Mesnager type with a slot 2 mm (5/64″) in depth rounded off at the bottom, cut at right angles to the main faces of the wide plates.

A considerable drop in the resilience in the zone adjacent to the fracture, which confirms the indications given by the tensile tests. These drops in resilience amount respectively to 58 % and 59 %. The existence of a zone affected by cold hardening is thus demonstrated.

As regards the figures obtained from the test pieces taken from the unaffected zone, these are satisfactory and correspond to those which can be expected for mild steels of good average quality.

F. — MICROGRAPHIC EXAMINATIONS (1). — The specimens taken from the two boom plates, show ferrous structures of mild steel of fine and regular grain. (Fig. 11.)

No deformation of the grains due to cold hardening was noted, under the microscope, in the zone adjacent to the fracture by decohesion.

Examinations of the rivets have justified the following remarks :

Two rivets showed a noticeable elongation at the neck, indicating a satisfactory degree of plasticity. Their structure is fine and regular.

The third rivet showed a clean fracture at the neck and indicates the superheating which is well shown close to the fractured section. The fragility of the superheated zone has favoured the detachment of the head (Figs. 12 & 13).

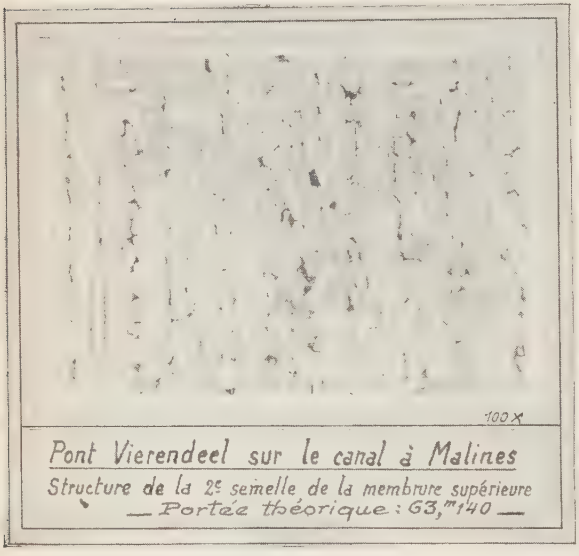
100 X
Pont Vierendeel sur le canal à Malines
Structure de la 2e semelle de la membrure supérieure
— Portée théorique : 63,m140 —

Fig. 11.

Vierendeel Bridge on the canal at Malines.
Structure of the 2nd. boom plate of the superstructure. Nominal span 63.140 m.

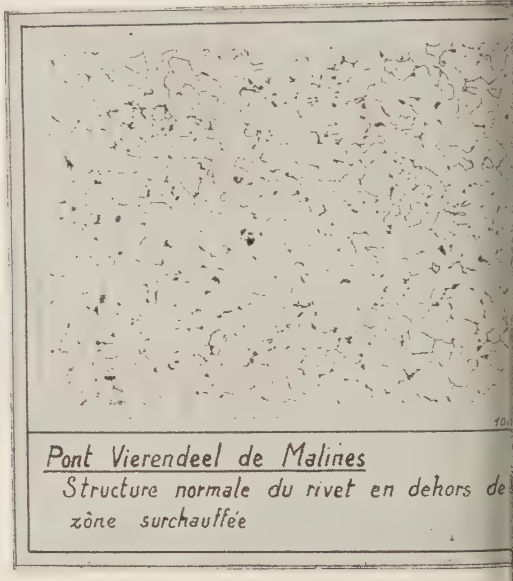

Fig. 12.

Vierendeel Bridge at Malines.
Normal structure of the rivet beyond the superheated zone.

(1) The original micrographs of Figs. 11, 12, 13 which were 100 × have been photographically reduced to 65 ×.

Fig. 13.

Vierendeel Bridge at Malines.

tructure of the superheated extremity of the rivet shank (at the head which was blown off by the explosion).

In conclusion it may be stated that the wo boom plates examined are mild steel f good quality which had not been killed.

This important structure was reconstructed after eliminating all the parts lestroyed, deformed or adjacent to the xplosions.

It was put back in service early in 1941.

***

The investigation was continued on other tructures and 41 superstructures, mostly lestroyed by dynamite in 1940; they were ubjected to tests of materials by means of ore holes in the zone adjacent to the xplosion and in the parts that were left ntact. Each superstructure examined was ubjected to a detailed report dealing with he anomalies observed, with conclusions rrived at for each case dealt with.

It is certainly not easy to separate from ontrols so restricted and frequently limited to a few detached parts, conclusions of an absolute character on the value of the materials employed in our rivetted structures, but we think we have kept within the limits of sound interpretation by setting up the following general considerations in support of which we have added the necessary comments.

1° Taken as a whole, the Thomas mild steels used in our riveted structures have very satisfactory characteristics which comply with the conditions prescribed by the specifications in force.

The diagrams below summarize the results obtained (Fig. 14).

As regards *tensile* stresses, it is to be noted that the highest percentages concern the elastic limits above 25 kg/mm² (15.87 t per sq. inch) and the breaking stresses exceed 35 kg/mm² (22.22 t per sq. inch). In both cases, the highest figures refer to semi-hard steels that are met with exceptionally.

We would also point out that the plastic stretching was of great importance and that over 81 % of the tests exceeded 26 % in the lengthwise elongation; while the crosswise stretchings exceeded 21 % of elongation and attained a recurrence of more than 73 %.

If we revert for a moment to the figures obtained with the iron, we find that for this material 90 % of the lengthwise elongations were less than 21 %.

2° The *chemical analysis* have sometimes revealed average contents in sulphur and phosphorus higher than the maxima usually admitted in the industry for the Thomas (basic) steels of current quality, e. g. Sulphur : 0.06 %; phosphorus : 0.075 %; Sulphur + Phosphorus : 0.12 %.

The mean maximum contents recorded exceptionally in the course of the analyses were as follows :

Sulphur : 0.097 %; phosphorus : 0.14 %.

These content figures are evidently too high and would tend to seriously damage the properties of the metal. Certain cracked web plates showed 0.14 % of phosphorus.

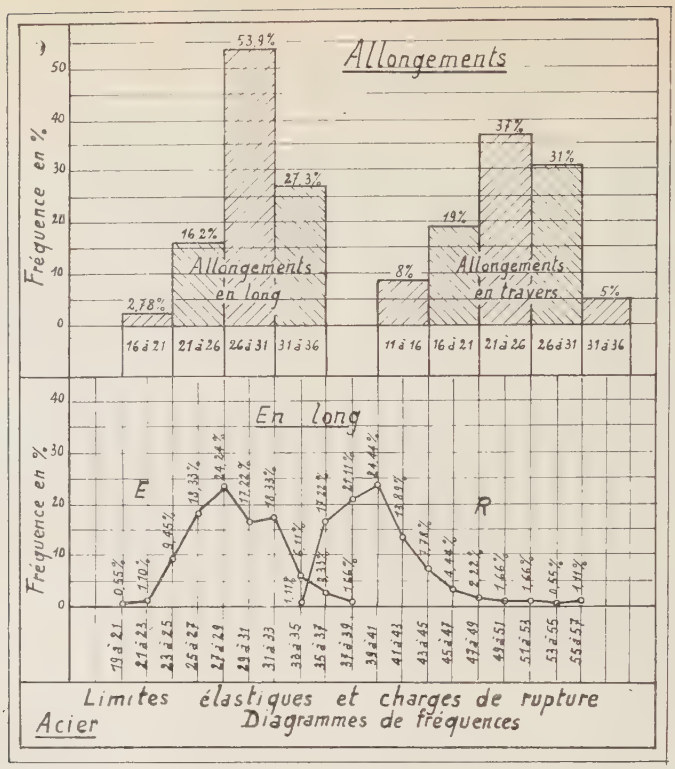

Fig. 14.

Explanation of French terms :

Limites élastiques et charges de rupture = Elastic limits and breaking loads. — Diagrammes de fréquences = Diagrams of frequencies. — Acier = Steel. = Allongements = Elongations. — Fréquence = Frequency. — En long = Lengthwise. — En travers = Crosswise.

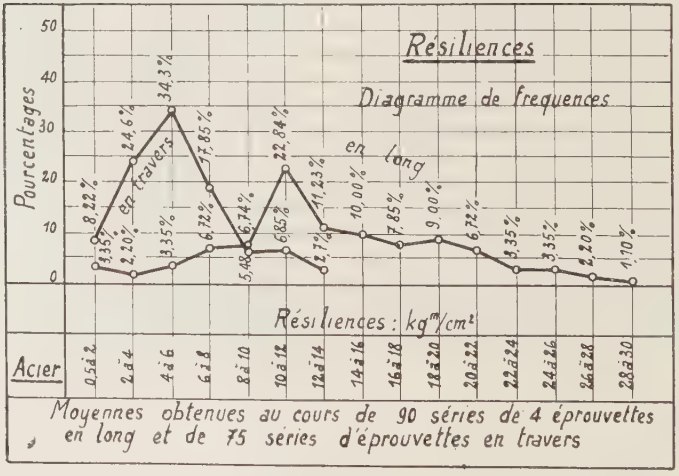

Fig. 15.

Explanation of French terms :

Acier = Steel. — Diagrammes de fréquences = Diagrams of frequencies. — Pourcentages = Percentages. — En long = Lengthwise. — Moyennes obtenues au cours de 90 séries de 4 éprouvettes en long et de 75 séries d'éprouvettes en travers. — Averages obtained with 90 sets of 4 test pieces in the longitudinal section and 75 sets of test pieces in cross section.

It must not be lost sight of that the ontents cited as a rule are averages of nalyses made from cuttings taken from he entire section of the rolled metal. The ctual contents in sulphur and phosphorus ould be much higher, if the measurements ere restricted to zones segregated from teels that had not been killed.

The metallurgists have nowadays an asy and economical way of desulphurizing ne liquid castings for conversion by sodium arbonate. This process is in general use nd it is possible in the ordinary course y treating these castings, to obtain steels basic) showing very low sulphur contents. $< 0.06\ \%$).

3° The *resilience tests*, not for use as cceptance tests for the material, have nabled a determination to be made of the eneral extent to which steels which have ot been killed, are able to resist failure of otched bars when subjected to mechanical nock.

The graph, Fig. 15, shows average results btained with the set of 4 test pieces selected om among the samples. From this it is en that 84.38 % of the lengthwise series ave figures lying between 8 and more than 8 kgm/cm².

When we get high resilience, e. g. 18 kgm, often happens with mild steels which show igh plastic deformation, that the test ieces bend without fracturing completely nd the test piece escapes the blow by assing down the passage left clear for the endulum. In such case, the figures for silience fail to indicate the total amount work which should be absorbed by the st piece.

We must remember that temperature as great influence on the fragility of the etal.

Fig. 16 shows generally the variation of silience in a mild steel with the tempe- ture ruling at the instant of test.

Point C of the curve shows tests made at)° C, which is the normal temperature of e air in the laboratory, within $\pm$ 5°.

If the temperature falls to near 0°, the op in resilience is sudden and remains very low at the low temperatures, especially for crude steels of ordinary quality from the rolls.

The phenomenon is of general occurrence and refers to all grades and qualities of steels.

On the curve, we note towards 500° (point E) a flexure which corresponds to the temperature of a blue colour, which is well-known by steel workers for the fragility which accompanies it.

Point F is the peak of the curve and is followed by a new drop corresponding to the beginning of viscous deformation due to the temperature reached.

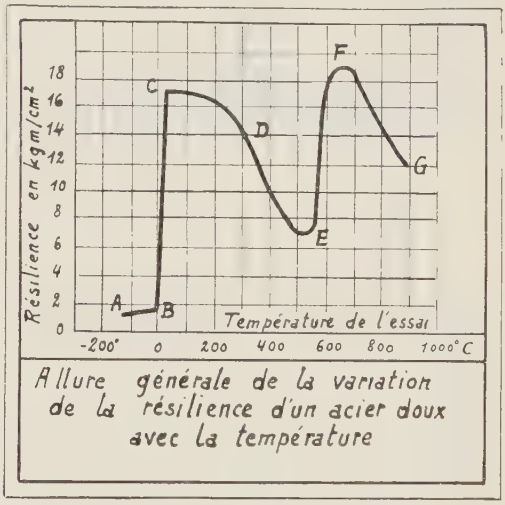

Fig. 16.

General view of variation of resilience of a mild steel with the temperature.

In general, the resilience tests made on samples of plates both flats and standard rolled sections, have not shown great dispersion which would hinder the appreciation of the very satisfactory averages obtained. This is largely due to the fact that we have only rarely come across marked segregation and few cases of coarse crystallization due to superheating.

Fig. 17.

The heaviest dispersions in the resilience values were seen in the heavy sections : broad-flanged grey girders, or ordinary beams of heavy section, in Thomas (basic) steel.

The fact must be borne in mind, since we have already noted the importance of the greater segregation which increases with the cross-section of the ingots. Certain heavy sections require ingots weighing up to 16 tons. The importance of the segregations in these sections are clearly seen in the macrographs and show the considerable chemical heterogeneity which results. In Fig. 17 we give an example of a Baumann macrograph taken from girders of large dimensions, obtained from heavy ingots and showing the extent of the segregation of sulphur.

If we add the very real difficulties in heating such ingots to an oven temperature before rolling, we find that in addition to the chemical heterogeneity we are faced with physical heterogeneity and that explains the great dispersion in the values obtained for resilience.

In addition, if we take into account the disturbed balance in the distribution of the masses between the web and the flanges of these heavy sections, an out-of-balance which results, owing to cooling after rolling, in contraction stresses which are both severe and unequal, capable of bringing about sudden fracture, one is bound to admit that these original loads have a bad effect on the quality of the material and may even cause unreliability and failure.

What we have just said about the heterogeneity of heavy rolled sections holds equally for heavy plates of large dimensions. This is the reason we are in favour of using killed, mild steel, with macrographic control and using resilience tests, for the plates forming the webs of important riveted superstructures having solid webs.

These practical tests made in carefully predetermined cases, would reveal the chemical and physical heterogeneity of the metal. They would be restricted to plates of a thickness exceeding 12 mm (15/32″),

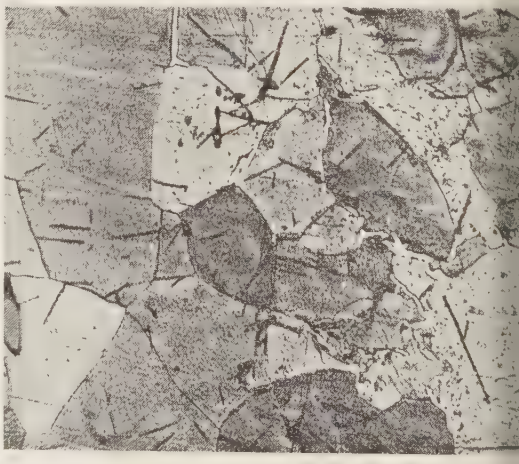

Fig. 18.

Ageing of the steels.

Thomas (basic) steel at 0,07 % C cold hardened to 5 % and heated to 200° for 30 min. Precipitation of nitride and intergranular cementite.

75 ×

Fig. 19.

Ageing of the steels.

Thomas (basic) steel cold hardened to 5 % followed by heating to 100° C for 1 hour

Needles of Nitrite.

« Attaque de Fry ».

he effervescence of the steel being admitted, nder the usual conditions provided for by the specifications, for plates of 12 mm or less, for wide plates and all standard sections.

We must still say a word about rivets for which mild steel not previously killed had been accepted hitherto.

It seemed to be necessary to supply these n killed steel, so as to reduce the risks of change by ageing and to avoid the formation of craters at the heads — a defect which appeared to be principally due to rapid burning of the metal in the central segregated portion of the shank when the rivet is heated.

There are evidently other causes producing alteration in the metal.

It is possible in effect to burn or overheat a rivet in the furnace by using a badly regulated arrangement (too high amperage) or by neglect (lack of cleanliness in the holes) or by leaving the rivet in any sort of furnace under a high temperature.

4° The hardening due to cold deformation : i. e. straightening, bending or hammering may, according to the weather and especially if the metal is heated moderately to a few hundred degrees, profoundly change the properties of steels by ageing, more especially in the case of steels which have not been killed.

Fig. 18 to 20 illustrate the theory of ageing which we have sketched out briefly in regard to iron. They show steels which after hardening and moderate heating, indicate nitrite precipitation.

We note also that hardening due to fatigue may simply give rise to slipping of the grain, which is shown in micrographic examination by the characteristic hemitropic structure (« macles »).

The nitrite precipitation only appears in the microscope if the nitrogen content is relatively high ($> 0.03\ \%$ according to Sauveur). Generally speaking, the mild Thomas steels produced by our steel works contain between $0.010\ \%$ and $0.018\ \%$ nitrogen.

Professor Lefebvre, in his remarkable paper on « Problems of Steel-making » read in November 1947, to the « Société

150 ×

Fig. 20.

High pressure flanged pipe hardened following alternating stresses.

Slipping lines (macles) in the grain.

Royale Belge des Ingénieurs et des Industriels », recalled the very pertinent fact, that in the course of preparing steels which have not been killed, the nitrogen content increases with the oxidisation period i. e. in proportion to the cubic metres of air blown in under pressure, with the average temperature of the operation and with the depth of metal in the converter.

By applying these data we can explain the least tendency towards ageing in puddled iron from the fact that the temperature required for producing iron in the pasty condition is lower than for the steel and that the quantity of air introduced is much more limited, thus reducing the solubility of the nitrogen.

This explanation is confirmed by the percentages of nitrogen found in the specimens of puddled iron taken from certain bridges that were examined.

The results of the analyses of the iron are given in the table (VII) below, where are likewise shown those referring to Siemens-Martin and to Thomas steels in order to enable comparisons to be made.

From these various considerations, it is evident that Thomas steel prepared as it is to-day, is more subject to hardening and ageing than puddled iron and than the steels obtained under other processes.

It is important therefore to improve the quality by making every effort compatible with the means that are at the steel makers disposal, to reduce to a minimum, the contents in oxygen, hydrogen and nitrogen, all elements which have a direct specific action on the final properties of the steel.

In riveted structures of mild steel which has not been killed, the changes due to ageing must be checked by means of certain precautions which form part of the best practice in structural work. Every operation, whether of cold or hot working which might lead to a hardening or tempering effect must be forbidden.

In the course of our investigations, some cases of ageing have been revealed in certain steel superstructures, thanks to the usual indications : increase of the elastic limit, of the breaking stress and drop in resilience and elongation. These cases were mostly met with in the parts which had suffered from explosive action and were not used again in the work of reconstruction. In some other cases, ageing was revealed in parts which were in good condition, especially in joint connections close to the explosion point; in every case these parts were removed. Ageing in these cases was either the effect of the explosion or of the fall between the abutments of the parts of the structure.

TABLE VII.

NITROGEN CONTENT.

Puddled irons	Siemens-Martin Steels	Thomas Steels
Between 0.0028 % and 0.0084 %	Between 0.0041 % and 0.0063 %	Between 0.010 % and 0.018 %

What is important to note is that the nitrogen contents of the iron are generally noticeably lower than those found in the Thomas steels.

In other respects, the comparative resilience tests made on bars of iron and steel, at Malines in 1939, in their natural state and artificially aged (elongations of 5 % and heating to 250° for 30 minutes) gave the same indications of the diminished tendency to ageing in the case of puddled iron.

5° In addition to the classic tests which have been mentioned, we made some *endurance* tests by repeated impacts.

These tests which lasted a long time were made on a limited number when the thickness of the samples made it possible to select the necessary test pieces.

For this sort of test it is helpful to give particulars of the characteristics of the Amsler machine which we used at Malines.

The test is made on a cylindrical test piece of 12 mm (15/32") dia., without notch and 190 mm (7 1/2") long, resting freely on supports spaced 170 mm (6 11/16") apart.

The test piece is subjected to blows from two 3 kg drop hammers, alternately striking two generators of opposite polarity.

The height of drop is 4 cm; thus each hammer gives a blow of 12 kgcm.

Each hammer delivers 60 blows per minute so that the test pieces receive 120 blows per minute.

The machine stops automatically when the test piece breaks or bends enough to operate the disconnector gear.

The test pieces selected lengthwise in mild steel which had not been killed resulted in figures of between 45 316 and 56 456 single blows.

The same tests on test pieces of puddled iron gave figures varying between 18 000 and 53 000 single blows.

As regards the iron, the test pieces often split longitudinally on a line corresponding to the neutral fibres and in the zone acted on by the blows. This slitting or cracking precedes the appearance of transverse fissures, which when they arise, do not always occur in line with the points of impact, but rather where an accumulation of superficial slag inclusions are found.

As to the steels, the cracking which starts at the point of origin of the fracture or causes sufficient bending to stop the machine, is normally cross-sectional and this crack starts in a line with one of the two points of impact, at the point where progressive hardening arises.

6° As regards *corrosion* of the steels, this is undoubtedly more pronounced than that observed with the irons; and numerous pieces of steel have reached us, which at certain points showed reductions of section of from 30 % to 40 %.

***

This enquiry has enabled us to set up a comparison between the iron and the steel and to set up, purely objectively, the balance between advantages and defects of these two materials — a balance which has led us to consider the qualities of the irons used in the construction of our older structures, as dangerous, with very few exceptions.

As regards the steels if we are faced with the question : « What can we say against (mild) Thomas steels, that have not been killed, of which the majority of riveted metal bridges on our railway are made? », our reply would be « nothing or almost nothing », based on the results which we have been discussing.

Summing up :

In service : no failure, barring a few cracks in those parts where there is a high percentage of Sulphur and Phosphorus.

Under test : a few very rare cases of change due to ageing, which it is possible to prevent by care in manufacture of the metal, which should tend to limit gaseous inclusions : viz. oxygen, hydrogen and particularly nitrogen, and by taking steps which should prevent hardening and tempering; also some very rare cases of fragility resulting partly from chemical heterogeneity due to very noticeable major segregation and in part to coarse crystalization arising from overheating of the metal; this last named cause of fragility not being restricted to Thomas steels, but certainly being common to all grades and qualities of steels.

We have also drawn attention to the heterogeneity arising from use of heavy rolled sections and thick plates of large dimensions due to the use of heavy ingots

(5 to 16 tons). Thus the engineer responsible for arranging the work will be warned and will decide the most suitable application indicated for each particular case.

In brief and taken as a whole, the position is a reassuring one as regards the behaviour of the Thomas steels used in our riveted structures.

It is on this optimistic opinion that we would take our stand in closing this enquiry, in spite of the fact that practical improvements in quality, where possible, are called for.

May this synthesis that we have tried to make, based on the results of tests, help to solve the problems raised by the quality of materials and lead to more courageous and more perfect action in the reconstruction of our unhappy country, which after 4 years of oppression, should soon recover the normal course of its laborious destinies under the banner of victory and liberty.

[625 .13]

Calculation of the dimensions of the masonry lining of tunnels,

by Luigi LAMAGNA,

Engineer, Professor at the Genoa University.

The calculation of the dimensions of ıe masonry lining of a tunnel presupposes knowledge of the axial curve of its cross ction, as also of the pressures exerted by s surrounding earth.

In the case of tunnels excavated at a allow depth below the free surface of ıe soil, and where consequently the hole of the superincumbent soil is ıpported by the lining, the theories ncerning pressure of the earth are fficient to determine the external forces ı this lining. In this case, and on the rticular assumption that the free surface the soil is inclined transversely, the stribution of pressure on the lining is ymmetrical and therefore there is no ed to make a study of the general haracter of the problem (*).

When the tunnels are dug at a great pth, the determination of the pressures ı the masonry lining has been the bject of study and research by numerous ıthors, who have assumed the soil to be mogeneous, coherent or non-adhesive. this connection we refer to the studies RITTER, KOMMERELL and BIERBAUMER r the coherent soils, those of CORINI d our own writers for loose ground.

(*) *Cf.* Ing Dario MARTINI : « Le gallerie rietali », *Ingegneria Ferroviaria*, October 1946.

The assumption of a homogeneous soil is necessary to enable the problem to be put in a simple form for purposes of calculation. According to the opinion generally held to-day, the external stresses on the lining may be simplified to consideration of the effect due to a mass of homogeneous material having a symmetrical cylindrical surface with reference to the axial vertical plan of the tunnel of which the generatrices are parallel to the said axis and of which the directrix is formed by a curve AHB and by two straight lines AD and BC inclined to the vertical (fig. 1) by $45^{o} - \frac{\varphi}{2}$. The extremities A and B of the curve lie on the horizontal tangent at the key O_1 of the extrados of the masonry lining.

According to the different authors that have worked on the problem, the curve AHB is either a parabola or a semi-ellipse, which actually differ very little, one from the other.

The rise $O_1H = f$ of the curve has different values in the different calculations and according to whether the soil is coherent or not.

When it is a question of a shallow tunnel, but dug in ground where the free surface is horizontal, the curve AHB does

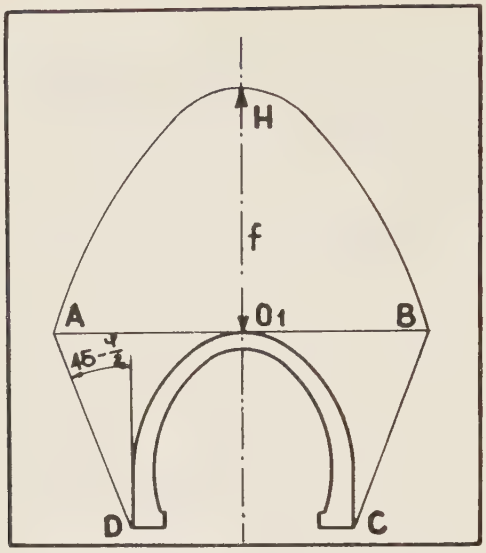

Fig. 1.

not exist and one should treat the straight lines DA and CB as being continued up to the free surface.

In these last two cases the effects on the lining as also the shape and dimensions of the latter, are symmetrical and problem of calculating them can be treated from a general aspect.

The calculation of the dimensions of the tunnel lining supposing it to have been dug to a great depth, has already been considered in a treatise which appeared a number of years ago, but it is not complete and contains inaccuracies which induced us to review the problem and to publish the present note, the object of which is to provide the analytic calculation of the thicknesses of the lining of a tunnel dug at a great or at a lesser depth below the surface of the ground, on the assumption that the stresses on this lining are symmetrical. It may be stated that the calculation can be made, based on known methods in the theory of elasticity; the proposed method has the advantage of using very simple formulae for resolution which make it possible to solve the problem quickly and readily.

General statement of the problem.

In general, the masonry linings of tunnels are designed so that we can examine the arch and the pillars separately. We shall accordingly start by considering the stability of the arch.

Let us consider the equilibrium of the left-hand half and substitute for the other the reaction which this exerts on the part that is retained, such reaction can be represented by a horizontal pressure H, in view of the supposed symmetry, and by a moment M_o.

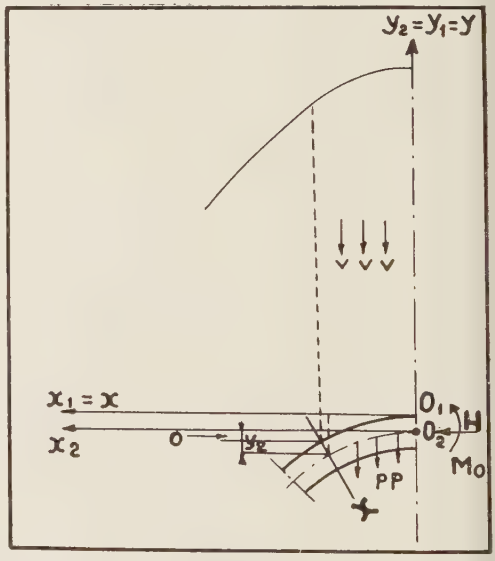

Fig. 2.

The theory of the derived function of work gives us the following equations :

$$\frac{\partial L}{\partial M_o} = 0 \qquad \frac{\partial L}{\partial H} = 0 \quad \ldots\ldots \quad (1)$$

where L = the work of deformation of the demi-arch in question, is expressed by :

$$L = \int \frac{N^2}{EF}\, ds + \int \frac{M^2}{EJ}\, ds + \int \chi \frac{T^2}{GF}\, ds \quad (2)$$

giving the symbols their usual signification.

If, as is usual, we neglect the effect of the normal stress N and of the shearing stress T, the second member of the equation (2) is reduced to its second term alone; the equations (1) then become :

$$\int M \frac{\partial M}{\partial M_o} \frac{ds}{EJ} = 0 \qquad \int M \frac{\partial M}{\partial H} \frac{ds}{EJ} = 0 \quad (3)$$

If we plot the masonry lining on two axes $x_2 y_2$, having their origin at the centre of gravity O_2 of the section of the key and arranged as in figure 2, y_2 will be the ordinate of the centre of gravity of a particular section X; hence if we resolve the forces in to their vertical and horizontal components, the bending moment in section X will be :

$$M = M_o - H y_2 + \mathfrak{M}_v + \mathfrak{M}_o + \mathfrak{M}_{pp} \quad (4)$$

in which $\mathfrak{M}_v$, $\mathfrak{M}_o$ and $\mathfrak{M}_{pp}$ are the moments, taken with respect to the centre of gravity of section X in question, the vertical and horizontal components of the stresses as also of the inherent weight of the masonry lining situated between the section X and the keystone. The signs of these moments will follow conventional practice.

In deriving the condition (4) with respect to M_o and to H, we have :

$$\frac{\partial M}{\partial M_o} = 1 \qquad \frac{\partial M}{\partial H} = -y_2$$

and consequently the relations (3) become :

$$\int \frac{M}{EJ}\, ds = 0 \qquad \int \frac{M y_2}{EJ}\, ds = 0 \quad (5)$$

equations which implicitly contain the parameters H and M_o and therefore serve to determine them. By referring to (4) and remembering that E is constant for the whole arch, we have finally :

$$\begin{aligned} \int \frac{1}{J} (M_o - H y_2 + \mathfrak{M}_v + \mathfrak{M}_o + \mathfrak{M}_{pp})\, ds = 0 \\ \int \frac{1}{J} (M_o - H y_2 + \mathfrak{M}_v + \mathfrak{M}_o + \mathfrak{M}_{pp})\, y_2\, ds = 0 \end{aligned} \quad (6)$$

the integrals being understood as extending over the whole left half of the arch.

From the equations (6), we gather that :

$$\left.\begin{aligned} M_o\int\frac{ds}{J} - H\int\frac{y_2}{J}\,ds &= -\int\frac{\mathfrak{M}_v}{J}\,ds - \int\frac{\mathfrak{M}_o}{J}\,ds - \int\frac{\mathfrak{M}_{pp}}{J}\,ds \\ M_o\int\frac{y_2}{J}\,ds - H\int\frac{y_2^2}{J}\,ds &= -\int\frac{\mathfrak{M}_v y_2}{J}\,ds - \int\frac{\mathfrak{M}_o y_2}{J}\,ds - \int\frac{\mathfrak{M}_{pp} y_2}{J}\,ds \end{aligned}\right\} \quad (6')$$

and by writing simply :

$$\int\frac{ds}{J} = I_m;\ \int\frac{y_2}{J}\,ds = I_h;\ \int\frac{\mathfrak{M}_v}{J}\,ds = I_v;\ \int\frac{\mathfrak{M}_o}{J}\,ds = I_o;\ \int\frac{\mathfrak{M}_{pp}}{J}\,ds = I_{pp};$$

$$\int\frac{y_2}{J}\,ds = I'_m;\ \int\frac{y^2_2}{I}\,ds = I'_h;\ \int\frac{\mathfrak{M}_v y_2}{J}\,ds = I'_v;\ \int\frac{\mathfrak{M}_o y_2}{J}\,ds = I'_o;\ \int\frac{\mathfrak{M}_{pp} y_2}{J}\,ds = I'_{pp}$$

the relations (6') become :

$$\left.\begin{aligned} M_o I_m - H I_h &= -I_v - I_o - I_{pp} \\ M_o I'_m - H I'_h &= -I'_v - I'_o - I'_{pp} \end{aligned}\right\} \quad (6'')$$

In solving by KRAMER's rule, by henceforward simply indicating by M the moment at the keystone of the arch and by indicating as follows the different determinants to be calculated, we get successively :

$$\Delta = \begin{vmatrix} I_m & I_h \\ I'_m & I'_h \end{vmatrix}$$

$$\Delta^m_v = \begin{vmatrix} I_v & I_h \\ I'_v & I'_h \end{vmatrix} \qquad \Delta^m_o = \begin{vmatrix} I_o & I_h \\ I'_o & I'_h \end{vmatrix} \qquad \Delta^m_{pp} = \begin{vmatrix} I_{pp} & I_h \\ I'_{pp} & I'_h \end{vmatrix}$$

$$\Delta^h_v = \begin{vmatrix} I_m & I_v \\ I'_m & I'_v \end{vmatrix} \qquad \Delta^h_o = \begin{vmatrix} I_m & I_o \\ I'_m & I'_o \end{vmatrix} \qquad \Delta^h_{pp} = \begin{vmatrix} I_o & I_{pp} \\ I'_o & I'_{pp} \end{vmatrix}$$

$$\left.\begin{aligned} M &= -\frac{\Delta^m_v}{\Delta} - \frac{\Delta^m_o}{\Delta} - \frac{\Delta^m_{pp}}{\Delta} \\ H &= \frac{\Delta^h_v}{\Delta} + \frac{\Delta^h_o}{\Delta} + \frac{\Delta^h_{pp}}{\Delta} \end{aligned}\right\}$$

and writing :

$$-\frac{\Delta_v^m}{\Delta} = M_v \qquad -\frac{\Delta_o^m}{\Delta} = M_o \qquad -\frac{\Delta_{pp}^m}{\Delta} = M_{pp}$$

$$\frac{\Delta_v^h}{\Delta} = H_v \qquad \frac{\Delta_o^h}{\Delta} = H_o \qquad \frac{\Delta_{pp}^h}{\Delta} = H_{pp}$$

we finally get :

$$\left.\begin{aligned} M &= M_v + M_o + M_{pp} \\ H &= H_v + H_o + H_{pp} \end{aligned}\right\} \qquad (7)$$

which show that one can take into account separately, in determining the parameters M and H, the only vertical components, the only horizontal components and finally the inherent weight of the masonry, a matter that is well known elsewhere and which is expressed by the principle of the superimposition of the loads.

In fact, the best way is to follow up this method which leads to a simplification of the whole of the calculations.

Moreover, by working on these lines it is possible to treat in the same way, the different cases which are presented according to whether the action of horizontal stresses or that of the inherent weight, or of both combined can be neglected with respect to the vertical stresses. In the case of tunnels dug in soils of very irregular consistencies, having a very small internal angle of friction φ, the effect of horizontal stresses will generally speaking not be negligible with respect to the vertical loads; the contrary effect is observed if the angle φ is relatively large, the soil being more compact. The effect of the inherent weight will be negligible or otherwise according to the height of the mass stressing the lining.

The shapes of the arch normally employed are the basket handle curve for railway tunnels and the circular curve for road tunnels. As regards the calculations which we propose to develop, the basket handle curves can be treated with an approximately satisfactory method by semi-ellipses of which the major axis will be vertical or horizontal according to whether we have to deal with single or double line tracks, or even by using parabolic arcs. We shall consider therefore, three cases, according to whether the axial curve is parabolic, elliptical or circular.

The thickness of the lining is very often constant, but it may also vary, either continuously or in stages. We shall consider the case of constant thickness and that of thickness with a continuous variation following the law :

$$J = J_o \frac{ds}{dx} \quad . \quad . \quad . \quad . \quad . \quad (8)$$

these are the only ones allowing of a sufficiently simple analytical study.

The relationship (8) can also be written :

$$\left(\frac{S_x}{S_o}\right)^3 = \frac{ds}{dx} = \sqrt{1 + \left(\frac{dy}{dx}\right)^2}$$

from which we deduce the law of variation of thicknesses :

$$s_x = s_o \sqrt[6]{1 + \left(\frac{dy}{dx}\right)^2} \qquad (8')$$

One can plot the geometric features of the lining and of the curve delimiting the mass which operates along the systems of axes shown in figure 2, i. e.

System $O_2\ x_2\ y_2$ for the directrix of the axial curve of the arch,

System $O_1\ x_1\ y_1$ (axis y_1 = axis y_2) for the directrix of the mass stressing the lining,

System $O_1\ x\ y$ (coinciding with $O_1\ x_1\ y_1$) for the directrix of the extrados of the arch.

In addition we shall introduce the adjustable (or variable) coordinates ξ_1, η_1 and $\xi\ \eta$ for indicating the general features for plotting in the second or in the third system of coordinate axes.

Finally let :

S_o be the half thickness of the arch at the keystone,

γ_t be the specific gravity of the soil,

γ_m be the specific gravity of the masonry forming the arch.

We shall demonstrate the numerous cases which may arise, from the simplest to the more complex (from the point of view of analytical development) following the order stated below :

A. *Surface tunnel (shallow).*

1. *Arch of variable thickness according to the relation* (8).

Axial curve of the arch :

a) parabolic;
b) elliptic;
c) circular.

2. *Arch of constant thickness.*

Axial curve of the arch :

a) parabolic;
b) elliptic;
c) circular.

B. *Deep tunnel.*

Subdivisions 1 — 2, *a*, *b* and *c* as for case A.

In order to shorten the calculations and to give the final results a sufficiently simple form and practical in application, we consider it opportune to introduce two simplifying hypotheses. The first consists in agreeing that the directrix of the extrados of the arch, which line forms the lower boundary of the loading mass, is always parabolic, even when the axial curve of this arch is elliptical or circular, which amounts to assuming that the roof which appears in the equation for the ellipse or the circle can be expanded into a series by the binomial theorem and that the expansion can be limited to the first two terms. From the practical point of view, this operation is always possible and does not introduce appreciable errors. The second, consists in agreeing that from the point of view alone of the inherent weight, the axial curve of the arch is always circular. We shall see that this allows of a considerable simplification of the calculations without however going beyond the normal limits for approximation.

A. *Case of surface (or shallow) tunnel.*

In this case, the external stresses on the lining of the arch are represented (fig. 3) by :

— Vertical stresses = weight of prism of section $D_1NMHM_1N_1C_1O_1D_1$ ($\times$ 1).

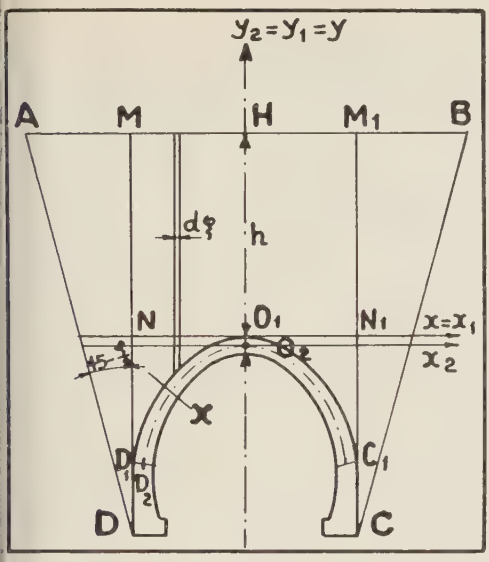

Fig. 3.

— Horizontal stresses = pressure of the bank of earth of height D_1N surcharged by a mass of earth of height $MN = h$ ($\times$ 1).

— Its inherent weight.

1. *General expression and particular expression for the moment* $\mathfrak{M}_v$.

If we represent by :

$$\eta_1 = \varphi_1(\xi) \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

the equation for the curve of the extrados, referred to the system of axes originating in O_1, the height of the mass which stresses it at the point of the abscissa ξ is :

$$h + \eta_1$$

and the elementary stress corresponding to an element $d\xi$ of this mass is :

$$\gamma_t(h + \eta_1)d\xi$$

The moment of this stress referred to the centre of gravity of a section X (fig. 3) of the abscissa $x_2 = x$ is :

$$d\mathfrak{M}_v = \gamma_t (h + \eta_1)(x - \xi)d\xi$$

and that of all the stresses operating to the right of this section, referred to the same point, is :

$$\mathfrak{M}_v = \gamma_t \int_o^x (h - \eta_1)(x - \xi)d\xi$$

$$= \gamma_t \left[h \frac{x^2}{2} + \int_o^x \eta_1 (x - \xi)\, d\xi \right] \quad . \quad . \quad (10)$$

If, as we have stated, we agree that the directrix of the extrados of the arch is a parabola having a rise H_1 and a half chord L_1 and hence of the equation :

$$\eta_1 = \frac{H_1}{L_1^2}\xi^2 = a_1\xi^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10')$$

we get :

$$\mathfrak{M}_v = \gamma_t \left[h \frac{x^2}{2} + a_1 \int_o^x \xi^2 (x - \xi)\, d\xi \right]$$

$$= \frac{\gamma_t}{12}(6\, hx^2 + a_1x^4) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

If the equation for the axial curve of the arch is :

$$y_2 = \frac{H_2}{L_2^2} x_2^2 = a_2x_2^2 \qquad \text{(see fig. 8)}$$

by writing :

$$\frac{a_1}{a_2} = k$$

the expression found (11) takes the final form :

$$\mathfrak{M}_v = \frac{\gamma_t}{12}(6\, hx^2 + ka_2x^4) \quad . \quad . \quad . \quad (11_A)$$

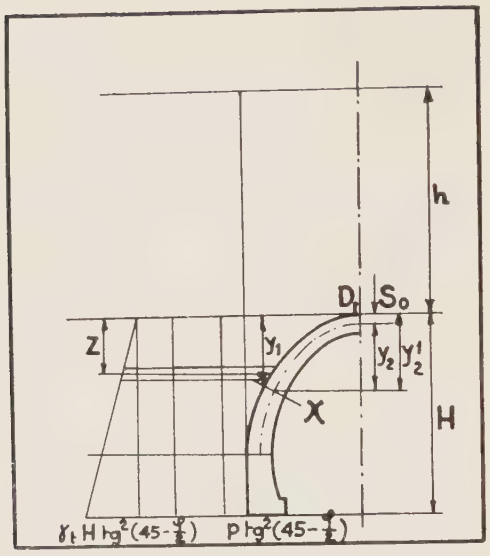

Fig. 4.

2. *General expression and particular expression for the moment* $\mathfrak{M}_o$.

The horizontal stresses are formed as we have already stated, by the pressure exerted by the earth bank limited as to its upper portion by the horizontal passing through O_1 and hence of height H (fig. 4), subject to the surcharge of a mass of height h.

Let us consider the two pressures separately, that of the bank of earth and that of the surcharge. The first at the generator depth z, amounts to :

$$\gamma_t z \operatorname{tg}^2\left(45^\circ - \frac{\varphi}{2}\right)$$

hence its pressure on a section of height dz is :

$$\gamma_t z \operatorname{tg}^2\left(45^\circ - \frac{\varphi}{2}\right) dz \quad \ldots\ldots (12)$$

The second has a constant value given by :

$$p \operatorname{tg}^2\left(45^\circ - \frac{\varphi}{2}\right) = \gamma_t h \operatorname{tg}^2\left(45^\circ - \frac{\varphi}{2}\right)$$

and consequently on the section of height dz :

$$\gamma_t h \operatorname{tg}^2\left(45^\circ - \frac{\varphi}{2}\right) dz \quad \ldots\ldots (13)$$

If we put :

$$\gamma_t \operatorname{tg}^2\left(45^\circ - \frac{\varphi}{2}\right) = \gamma'_t$$

and if we add (12) and (13), we have :

$$\gamma'_t (h + z)\, dz$$

The elementary moment referred to the centre of gravity of the section X is :

$$d\mathfrak{M}_o = \gamma'_t (h + z)(y'_2 - z)\, dz$$

and the moment of all the horizontal pressures acting to the right of section X with respect to the centre of gravity of this section :

$$\mathfrak{M}_o = \gamma'_t \int_o^{y_1} (h + z)(y'_2 - z)\, dz = \gamma'_t \left(h y'_2 y_1 - h \frac{y_1^2}{2} + y'_2 y_1^2 - \frac{y_1^3}{3} \right) \quad \ldots (14)$$

Supposing moreover that here the extrados had a parabolic directrix whose equation is : $y_1 = a_1 x^2 = k a_1 s^2$ and if we observe the figure 4 which shows that : $y'_1 = y_2 + s_o$.

The expression (14) will then become :

$$\mathfrak{M}_o = k a_2 \gamma'_t \left[h s_o x^2 + a_2 \left(s_o + \frac{h}{2} \right) x^4 + \frac{2}{3} a_2^2 x^6 \right] \quad \ldots\ldots (14_A)$$

3. *General expression for moment* $\mathfrak{M}_{pp}$.

By calling the thickness of the lining at the particular abscissa ξ by the symbol $2s_\xi$, the weight of an element ds of this lining is :

$$2\,\gamma_m\,s_\xi\,ds$$

its moment with reference to the centre of gravity of section X is then :

$$d\mathfrak{M}_{pp} = 2\,\gamma_m\,s_\xi\,(x-\xi)\,ds$$

and that of the whole section of the lining to the right of X :

$$\mathfrak{M}_{pp} = 2\gamma_m \int_o^x s_\xi\,(x-\xi)\,ds$$

and as moreover :

$$ds = \sqrt{1+\left(\frac{dy_2}{d\xi}\right)^2}\,d\xi$$

it follows that :

$$\mathfrak{M}_{pp} = 2\gamma_m \int_o^x s_\xi (x-\xi)\sqrt{1+\left(\frac{dy_2}{d\xi}\right)^2}\,d\xi \quad \ldots\ldots\ldots \quad (15)$$

As we are aware of the function s_ξ (law governing the variation of the thickness) and $y_2 = y_2\,(\xi)$ (equation for the axial curve of the arch), we possess all the necessary elements for calculating this moment. In the particular case of constant thickness and equal to $2\,s_o$, the relationship (15) becomes :

$$\mathfrak{M}_{pp} = 2\gamma_m s_o \int_o^x (x-\xi)\sqrt{1+\left(\frac{dy_2}{d\xi}\right)^2}\,d\xi. \quad \ldots\ldots\ldots \quad (15')$$

and where the thickness varies according to the law expressed in equation (8), we should have in virtue of (8') :

$$\mathfrak{M}_{pp} = 2\gamma_m s_o \int_o^x (x-\xi)\left[1+\left(\frac{dy_2}{d\xi}\right)^2\right]^{\frac{2}{3}}\,d\xi \quad \ldots\ldots\ldots \quad (15'')$$

We have already drawn attention to the fact that the effect of the inherent weight is generally very weak compared with that of the other vertical forces, so that it appears to be useless for all practical needs to calculate the integrals which are found in equations (15') and (15'').

Thus in the case of the arch of variable thickness, we can introduce a mean thickness of $2\,s_m$; equation (15'') will then take the form :

$$\mathfrak{M}_{pp} = 2\gamma_m s_m \int_o^x (x-\xi)\sqrt{1+\left(\frac{dy_2}{d\xi}\right)^2}\,d\xi \quad \ldots\ldots\ldots \quad (15''')$$

of the same pattern as (15').

We now see that the integral, which appears in this expression, is the static moment of the axial curve between the abscissae O and x, with respect to the vertical passing through the point of intersection. We may suppose that for purpose of calculation, that as we have stated, this curve is always circular; hence its static moment (fig. 5) amounts to :

$$m = 2\,R\,\alpha\,x - R\,(R - \sqrt{R^2 - x^2})$$

where :

R = radius of the circumference,

AB = 2 Rα, the development of the arc of angle at centre 2 α,

$$OG = \frac{R}{\alpha}\sin\,\alpha$$

$$GM = x - GN = x - \frac{R}{\alpha}\sin^2\,\alpha$$

$$\sin\,2\,\alpha = \frac{x}{R}$$

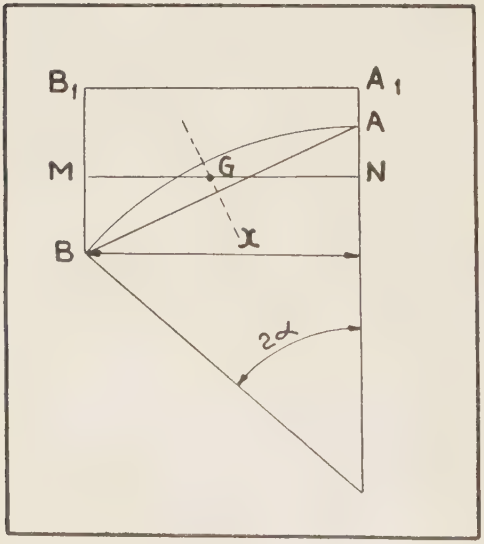

Fig. 5.

for $x = O$, we have likewise $\alpha = O$ and hence $m = O$ and for $x = R$, $\alpha = \frac{\pi}{4}$ and as a result $m = 0.57\ R^2$.

Now let us consider the segment A_1B_1 normal to OA and equal in length to x.

Its static moment with reference to the extremity B_1 of abscissa x is $0.50\ x^2$ and when the segment has the same length as the radius of the circumference, this value becomes $0.50\ R^2$. Hence, in the case of the tunnel linings, the ratio between the rise and the half chord of the arch does not differ greatly from unity and consequently, one can take it by approximation as equal to 1. In these conditions, when x varies from 0 to R, the static moment above this varies from 0 to $0.57\ R^2$. That of segment A_1B_1 varies in the same way from 0 to $0.50\ R^2$ and their ratio always varies by the same amount of 1 to 1.14. One can therefore, without committing too great an error, agree to a mean value of this ratio at 1.07. In other words, on the average, the static moment of the portion of the arc AB with respect to its extremity B of the abscissa x is equal to 1.07 times the moment of a horizontal segment of length x.

We may therefore write :

$$\mathfrak{M}_{pp} = 2\,\gamma_m\,s_o\,.\,1.07\,\frac{x^2}{2}$$

$$= 1.07\,\gamma_m\,s_o\,x^2 \quad . \quad . \quad . \quad . \quad . \quad (16_A)$$

for the case of constant thickness, and :

$$\mathfrak{M}_{pp} = 2\,\gamma_m\,s_m\,.\,1.07\,\frac{x^2}{2}$$

$$= 1.07\,\gamma_m\,s_m\,x^2 \quad . \quad . \quad . \quad . \quad . \quad (16_{A'})$$

for the case of variable thickness.

B. *Case of deep tunnel.*

In this case the external stresses on the lining of the arch are shown in figure 6 :

— vertical stresses = weight of the prism of section D_1 N M H M_1 N_1 C_1 O_1 D_1 (× 1).

— horizontal stresses = pressure of the earth bank of height D_1N subject to the surcharge of the mass of earth of section A N M (× 1).

— its inherent weight.

1. *General expression and particular expression of moment* $\mathfrak{M}_v$.

We assume that the directrix curve of the surface, which limits the mass (A M H M_1 B) acting on the arch is a parabola, and we call its rise *f*.

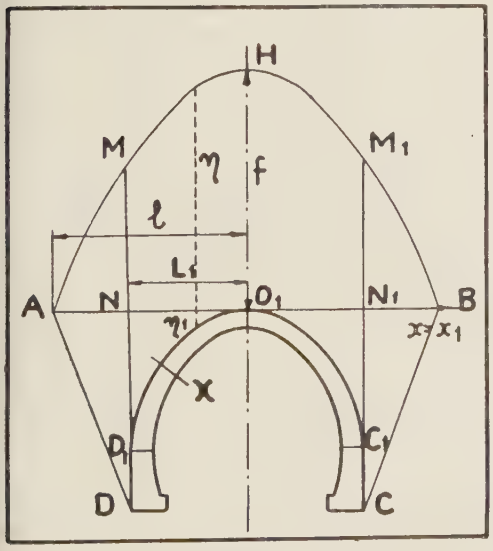

Fig. 6.

On the other hand let :

l be the semi-chord of this parabola and L_1 be the semi-chord of the curve of the extrados of the lining.

The equation of the parabola A M H M_1 B_1, referred to the axes arranged as shown in the figure is :

$$\eta = f - \frac{f}{l^2}\xi^2 = f - a\,\xi^2$$

by calling the particular abscissa ξ and putting :

$$\frac{f}{l^2} = a$$

As in other respects the equation of the curve of the extrados is :

$$\eta_1 = \frac{H_1}{L_1^2}\xi^2 = a_1\,\xi^2$$

as a result of equation (10′), we get :

$$\eta + \eta_1 = f + (a_1 - a)\,\xi^2$$

The elementary charge corresponding to an element d_ξ of the mass is :

$$\gamma_t\,[f + (a_1 - a)\,\xi^2]\,d\xi$$

The moment of the loads acting on the right of section X (fig. 6), referred to the centre of gravity of this section, is :

$$\mathfrak{M}_v = \gamma_t \int_o^x [f + (a_1 - a)\xi^2]\,(x - \xi)\,d\xi$$

$$= \frac{\gamma_t}{12}\,[6fx^2 + (a_1 - a)x^4]$$

and if we finally put :

$$\frac{a_1 - a}{a_2} = k_1$$

we get :

$$\mathfrak{M}_v = \frac{\gamma_t}{12} [6 f x^2 + k_1 a_2 x^4] \quad . \quad . \quad . \quad (11_B)$$

an expression which is perfectly analogous to (11_A) both in form and as regards its meaning.

2. *General expression and particular expression for moment* $\mathfrak{M}_o$.

The horizontal stresses are produced as we have already stated, by the pressure exerted by the earth bank limited to its upper part by the horizontal lead through O_1 and thus of a height H (fig. 7), subject to the surcharge from the mass of section A M N.

Similar considerations to those, which we have developed in dealing with case A, give us for the pressure on an element *dz* at the depth *z* below the horizontal through O_1 :

$$\gamma'_t (h + z)\, dz$$

where :

$$\gamma'_t = \gamma_t \operatorname{tg}^2 \left(45^o - \frac{\varphi}{2}\right)$$

$$h = \mathrm{MN} = f \left(1 - \frac{L_1^2}{l^2}\right)$$

Hence, the moment $\mathfrak{M}_o$ given by :

$$\mathfrak{M}_o = \gamma'_t \left(h y'_2 y_1 - h \frac{y_1^2}{2} + y'_2 y_1^2 - \frac{y'^3}{3}\right)$$

and assuming that one might assimilate y_1 and y_2, that y_1 needs not be distinguished from y_2 and by putting $y'_2 = y_2 + s_o$ as was done in case A, we get :

$$\mathfrak{M}_o = k\gamma'_t \left[h s_o a_2 x^2 + a_2^2 \left(s_o + \frac{h}{2}\right) x^4 + \frac{2}{3} a_2^3 x^6\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad (14_B)$$

exactly as in the preceding case, except as regards the meaning of *h*.

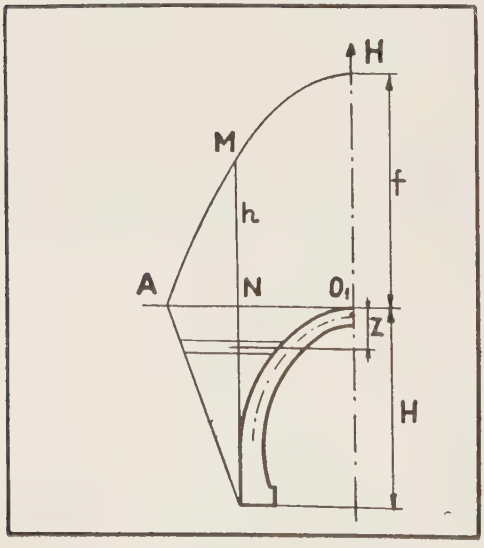

Fig. 7.

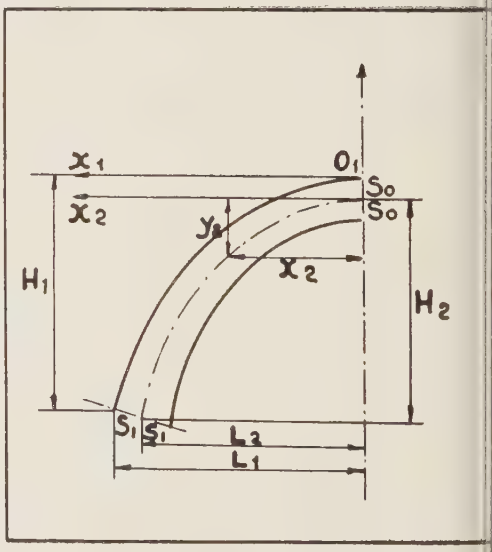

Fig. 8.

3. *General and particular expressions for moment* $\mathfrak{M}_{pp}$.

Here we can repeat what we have said in respect to case A; so that we have :

$\mathfrak{M}_{pp} = 1.07\ \gamma_m s_o x^2$ for the arch of constant thickness (16_B);

$\mathfrak{M}_{pp} = 1.07\ \gamma_m s_m x^2$ for the arch of variable thickness ($16'_B$).

We must now consider the different cases which arise when the thickness of the arch is variable or constant and according to the shape of the axial curve, as indicated above.

1. *Arch of variable thickness according to law* (8).

In the integrals which appear in equations (6') the fraction $\frac{ds}{J}$ becomes $\frac{dx}{J_o}$ and as J_o is constant and common to all the terms, we can omit it.

The different integrals to be calculated are then reduced to the following :

$$\int dx = I_m \qquad \int y_2 dx = I'_m$$

$$\int y_2 dx = I_h \qquad \int y_2^2 dx = I'_h$$

$$\int \mathfrak{M}_v dx = I_v \qquad \int \mathfrak{M}_v y_2 dx = I'_v$$

$$\int \mathfrak{M}_o dx = I_o \qquad \int \mathfrak{M}_o y_2 dx = I'_o$$

$$\int \mathfrak{M}_{pp} dx = I_{pp} \qquad \int \mathfrak{M}_{pp} y_2 dx = I'_{pp}$$

a) Arch with parabolic axial curve.

The equation of the axial curve, referred to axes arranged as in figure 8, is :

$$y_2 = \frac{H_2}{L_2^2} x_2^2 = a_2 x_2^2$$

Expression of the different integrals I :

$$I_m = \int_o^{L_2} dx = L_2$$

$$I_h = \int_o^{L_2} y_2 dx = \frac{H_2 L_2}{3}$$

$$I_v = \int_o^{L_2} \mathfrak{M}_v dx$$

$$\text{Case A} = \frac{\gamma_t}{12} L_2^3 \left(2\ h + k\ \frac{H_2}{5}\right)$$

$$\text{Case B} = \frac{\gamma_t}{12} L_2^3 \left(2\ f + k_1\ \frac{H_2}{5}\right)$$

$$I_o = \int_o^{L_2} \mathfrak{M}_o dx = \gamma'_t k L_2 H_2 \left[\frac{h s_o}{3} + \left(s_o + \frac{h}{2}\right)\frac{H_2}{5} + \frac{2}{21} H_2^2\right]$$

$$I_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} dx = 1.07 \gamma_m s_m \frac{L_2^3}{3}$$

$$I'_m = \int_o^{L_2} y_2 dx = \frac{H_2 L_2}{3}$$

$$I'_h = \int_o^{L_2} y_2^2 dx = \frac{H_2^2 L_2}{5}$$

$$I'_v = \int_o^{L} \mathfrak{M}_v y_2 dx$$

$$\text{Case A} = \frac{\gamma_t}{12} H_2 L_2^3 \left(\frac{6h}{5} + k\ \frac{H_2}{7}\right)$$

$$\text{Case B} = \frac{\gamma_t}{12} H_2 L_2^3 \left(\frac{6f}{5} + k_1\ \frac{H_2}{7}\right)$$

$$I'_o = \int_o^{L_2} \mathfrak{M}_o y_2 dx = \gamma'_t k H_2 L_2^2 \left[\frac{h s_o}{5} + \left(s_o + \frac{h}{2}\right)\frac{H_2}{7} + \frac{2}{27} H_2^2\right]$$

$$I'_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} y_2 dx = 1.07 \gamma_m s_m H_2 \frac{L_2^3}{5}$$

It will be noticed that all the integrals I are divisible by L_2 and all the integrals I' by $H_2 L_2$.

Knowing the values of I and I', the problem is now completely solved and the determination of the moment and of the thrusts depends solely on solving systems of linear equations with two unknown quantities.

Having in the meantime been given the very simple form of the final results we think it well to reproduce them below :

Case A :

$$
\left.
\begin{aligned}
M_v &= -\frac{1}{140}\,\gamma_t k H_2 L_2^2 \\
H_v &= \frac{1}{14}\,\gamma_t \frac{L_2^2}{H_2}(7h + kH_2) \\
M_o &= -\frac{3}{7}\,\gamma'_t k H_2^2 \left[\frac{1}{5}\left(s_o + \frac{h}{2}\right) + \frac{4}{27}H_2\right] \\
H_o &= 15\gamma'_t k \left[\frac{1}{15}hs_o + \frac{2}{35}\left(s_o + \frac{h}{2}\right)H_2 + \frac{2}{63}H_2^2\right] \\
M_{pp} &= 0 \\
H_{pp} &= 1.07\,\frac{\gamma_m s_m}{a_2}
\end{aligned}
\right\} \quad (17_A)
$$

Case B :

$$
\left.
\begin{aligned}
M_v &= -\frac{1}{140}\,\gamma_t k_1 H_2 L_2^2 \\
H_v &= \frac{1}{14}\gamma_t \frac{L_2^2}{H_2}(7f + k_1 H_2) \\
M_o &= -\frac{3}{7}\gamma'_t H_2^2 \left[\frac{1}{5}\left(s_o + \frac{h}{2}\right) + \frac{4}{27}H_2\right] \\
H_o &= 15\gamma'_t \left[\frac{1}{15}hs_o + \frac{2}{35}\left(s_o + \frac{h}{2}\right)H_2 + \frac{2}{63}H_2^2\right] \\
M_{pp} &= 0 \\
H_{pp} &= 1.07\,\frac{\gamma_m s_m}{a_2}
\end{aligned}
\right\} \quad (17_B)
$$

b) Arch with axial elliptic curve.

The equation for the axial curve of the arch, referred to the axes $x\ y_2$ as shown in figure 8, is :

$$y_2 = H_2\left(1 - \sqrt{1 - \frac{x^2}{L_2^2}}\right) \quad \ldots\ldots\ldots\ldots \quad (18)$$

By introducing the eccentricity of the ellipse given by

$$e^2 = \frac{L_2^2 - H_2^2}{L_2^2}, \text{ if } L_2 > H_2,$$

equation (18) then reads :

$$y_2 = L_2\sqrt{1 - e^2}\ \left(1 - \sqrt{1 - \frac{x^2}{L_2^2}}\right) \quad \ldots\ldots\ldots\ldots \quad (18')$$

if on the other hand $L_2 < H_2$, the eccentricity is : $e^2 = \dfrac{H_2^2 - L_2^2}{H_2^2}$

so that equation (18) becomes :

$$y_2 = \frac{L_2}{\sqrt{1 - e^2}}\left(1 - \sqrt{1 - \frac{x^2}{L_2^2}}\right) \quad \ldots\ldots\ldots\ldots \quad (18'')$$

By grouping (18') and (18'') to form a single expression, we can write

$$y_2 = L_2(1 - e^2)^{\pm\frac{1}{2}}\left(1 - \sqrt{1 - \frac{x^2}{L_2^2}}\right) \quad \ldots\ldots\ldots\ldots \quad (19)$$

be agreeing to use the higher or lower index according to whether $L_2 \gtrless H_2$.

For the moments $\mathfrak{M}_v$, $\mathfrak{M}_o$, $\mathfrak{M}_{pp}$, apply also the expressions (11_A), (11_B), (14_A), (14_B), (16_A), (16_B), $(16'_{A\ -\ B})$ which have already been seen.

The integrals I and I' are given by :

$$I_m = \int_o^{L_2} dx = L_2$$

$$I_h = \int_o^{L_2} y_2 dx = L_2(1 - e^2)^{\pm\frac{1}{2}}(L_2 - \Phi_o)$$

$$I_v = \int_o^{L_2} \mathfrak{M}_v dx =$$

$$\text{case A} = \frac{\gamma_t}{12} L_2{}^3 \left(2\ h + k\ \frac{H_2}{5}\right)$$

$$\text{case B} = \frac{\gamma_t}{12} L_2{}^3 \left(2\ f + k_1\ \frac{H_2}{5}\right)$$

$$I_o = \int_o^{L_2} \mathfrak{M}_o dx = \gamma'_t k H_2 L_2 \left[\frac{hs_o}{3} + \left(\frac{h}{2} + s_o\right)\frac{H_2}{5} - \frac{2}{21} H_2{}^2\right]$$

$$I_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} dx = 1.07 \gamma_m s_m \frac{L_2{}^3}{3}$$

$$I'_m = \int_o^{L_2} y_2 dx = L_2{}^2 (1 - e^2)^{\pm \frac{1}{2}} \left(1 - \frac{\Phi_o}{L_2}\right)$$

$$I'_h = \int_o^{L_2} y_2^2 dx = L_2{}^3 (1 - e^2)^{\pm 1} \left(\frac{5}{3} - 2\frac{\Phi_o}{L_2}\right)$$

$$I'_v = \int^{L_2}$$

$$\text{case A} = \frac{\gamma_t}{12} L_2{}^4 (1 - e^2)^{\pm \frac{1}{2}} \left[6\ h \left(\frac{1}{3} - \frac{\Phi_2}{L_2{}^3}\right) + k H_2 \left(\frac{1}{5} - \frac{\Phi_4}{L_2{}^5}\right)\right]$$

$$\text{case B} = \frac{\gamma_t}{12} L_2{}^4 (1 - e^2)^{\pm \frac{1}{2}} \left[6\ f \left(\frac{1}{3} - \frac{\Phi_2}{L_2{}^3}\right) + k_1 H_2 \left(\frac{1}{5} - \frac{\Phi_4}{L_2{}^2}\right)\right]$$

$$I'_o = \int_o^{L_2} \mathfrak{M}_o y_2 dx = k \gamma'_t H_2 L_2{}^2 (1 - e^2)^{\pm \frac{1}{2}} \left[hs_o \left(\frac{1}{3} - \frac{\Phi_2}{L_2{}^3}\right) + H_2 \left(s_o + \frac{h}{2}\right)\left(\frac{1}{5} - \frac{\Phi_4}{L_2{}^5}\right) + \frac{2}{3} H_2{}^2 \left(\frac{1}{7} - \frac{\Phi_6}{L_2{}^7}\right)\right]$$

$$I'_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} y_2 dx = 1.07 \gamma_m s_m L_2{}^4 (1 - e^2)^{\pm \frac{1}{2}} \left(\frac{1}{3} - \frac{\Phi_2}{L_2{}^3}\right)$$

writing :

$$\Phi_0 = \int_o^{L_2} \sqrt{1 - \frac{x^2}{L_2^2}}\, dx$$

$$\Phi_2 = \int_o^{L_2} x^2 \sqrt{1 - \frac{x^2}{L_2^2}}\, dx$$

$$\Phi_4 = \int_o^{L_2} x^4 \sqrt{1 - \frac{x^2}{L_2^2}}\, dx$$

$$\Phi_6 = \int_o^{L_2} x^6 \sqrt{1 - \frac{x^2}{L_2^2}}\, dx$$

The integrals Φ_0 Φ_2 Φ_4 Φ_6 are readily calculated, by putting :

$$\frac{x^2}{L_2^2} = \sin^2 \varphi$$

from which

$$\sqrt{1 - \frac{x^2}{L_2^2}} = \cos \varphi \text{ and } dx = L_2 \cos\varphi \; d\varphi$$

The arcs $0, \frac{\pi}{2}$ correspond to the limits of integration 0, L_2.

Thus, we shall get

$$\Phi_0 = L_2 \int_o^{\frac{\pi}{2}} \cos^2 \varphi d\varphi = \frac{\pi}{4} L_2$$

$$\Phi_2 = L_2^3 \int_o^{\frac{\pi}{2}} \cos^2 \varphi \sin^2 \varphi d\varphi = \frac{\pi}{16} L_2^2$$

$$\Phi_4 = L_2^5 \int_o^{\frac{\pi}{2}} \cos^2 \varphi \sin^4 \varphi d\varphi = \frac{\pi}{32} L_2^5$$

$$\Phi_6 = L_2^7 \int_o^{\frac{\pi}{2}} \cos^2 \varphi \sin^6 \varphi d\varphi = \frac{5\pi}{256} L_2^7$$

Taking into account these values and noting that the expressions for I are all divisible by L_2 and those for I′ by $L_2^2\,(1-e^2)^{\pm\frac{1}{2}}$, we get :

Case A :

$$
\left.
\begin{aligned}
M_v &= -\frac{\gamma_t}{100}\,L_2^2(2.5769\ h - 0.4518\ kH_2)\\
H_v &= \gamma_t L_2(0.6573\ h + 0.0985\ kH_2)\ (1-e^2)^{\mp\frac{1}{2}}\\
M_o &= -0.0522\gamma'_t kH_2\left[hs_o - H_2\left(s_o+\frac{h}{2}\right) - H_2^2\right]\\
H_o &= \gamma'_t k\ \frac{H_2}{L_2}\left[1.310\ hs_o + 1.178\ H_2\left(s_o+\frac{h}{2}\right) + 0.678\ H_2^2\right](1-e^2)^{\mp\frac{1}{2}}\\
M_{pp} &= -0.06145\gamma_m s_m L_2^2\\
H_{pp} &= \ \ 1.3916\gamma_m s_m L_2(1-e^2)^{\mp\frac{1}{2}}
\end{aligned}
\right\} \qquad (20_A)
$$

Case B :

$$
\left.
\begin{aligned}
M_v &= -\frac{\gamma_t}{100}\,L_2^2(2.5769\ f - 0.4518\ k_1H_2)\\
H_v &= \gamma_t L_2\,[0.6573\ f + 0.0985\ k_1H_2]\,(1-e^2)^{\mp\frac{1}{2}}\\
M_o &= -0.0522\gamma'_t kH_2\left[hs_o - H_2\left(s_o+\frac{h}{2}\right) - H_2^2\right]\\
H_o &= \gamma'_t k\ \frac{H_2}{L_2}\left[1.310\ hs_o + 1.178\left(s_o+\frac{h}{2}\right) + 0.678\ H_2^2\right](1-e^2)^{\mp\frac{1}{2}}\\
M_{pp} &= -0.06145\gamma_m s_m L_2^2\\
H_{pp} &= 1.3916\gamma_m s_m L_2(1-e^2)^{\mp\frac{1}{2}}
\end{aligned}
\right\} \qquad (20_B)
$$

Using the convention that the upper (+) signs refer to the ellipse in which $L_2 > H_2$ and the lower (—) signs to those where $L_2 < H_2$.

c) Arch with circular axial curve.

The formulae for solving the case of the circular arch can be deduced directly from the foregoing by putting $e = 0$.

In this way, we shall obtain two groups of formulae analagous to the formulae (20_A) and (20_B), in which the term $(1 - e^2)^{\mp \frac{1}{2}}$ will not appear.

2. *Arch of constant thickness.*

In the terms referring to the relations (6'), the divisor J is constant and can therefore be eliminated.

Knowing moreover that :

$$ds = \sqrt{1 + \left(\frac{dy}{dx}\right)^2}\, dx$$

the integrals I and I' take the form :

$$I_m = \int_o^{L_2} \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx \qquad I'_m = \int_o^{L_2} y_2 \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx$$

$$I_h = \int_o^{L_2} y_2 \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx \qquad I'_h = \int_o^{L_2} y_2^2 \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx$$

$$I_v = \int_o^{L_2} \mathfrak{M}_v \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx \qquad I'_v = \int_o^{L_2} \mathfrak{M}_v y_2 \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx$$

$$I_o = \int_o^{L_2} \mathfrak{M}_o \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx \qquad I'_o = \int_o^{L_2} \mathfrak{M}_o y_2 \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx$$

$$I_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx \qquad I'_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} y_2 \sqrt{1 + \left(\frac{dy}{dx}\right)^2} dx$$

We shall still examine here the three cases corresponding to the parabolic, elliptic and circular directrix, as also those referred to above as cases A and B concerning the surface and deep tunnels.

a) Arch with axial parabolic curve.

The equation of the axial curve, referred o coordinate axes as seen in figure 9, as ve have already seen, is :

$$y_2 = \frac{H_2}{L_2^2} x^2 = a_2 x^2$$

herefore :

$$\frac{dy}{dx} = 2a_2 x,$$

nd

$$ds = \sqrt{1 + 4a_2^2 x^2}\, dx$$

The integrals I and I′ become then :

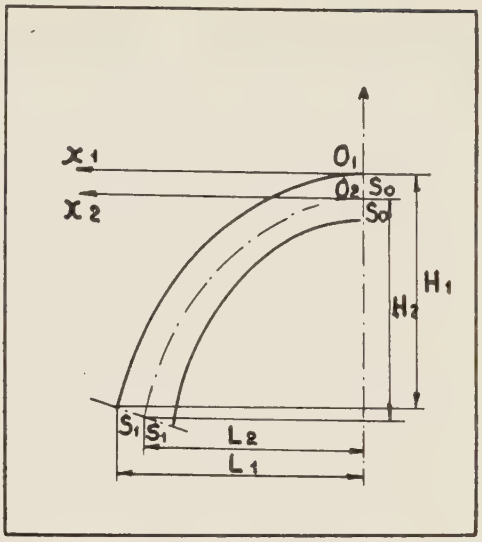

Fig. 9.

$$I_m = \int_o^{L_2} ds = \int_o^{L_2} \sqrt{1 + 4a_2^2 x^2}\, dx = \Psi_o$$

$$I_h = \int_o^{L_2} y_2 ds = a_2 \int_o^{L_2} x_2 \sqrt{1 + 4a_2^2 x^2}\, dx = a_2 \Psi_2$$

$$I_v = \int_o^{L_2} \mathfrak{M}_v ds =$$

$$\text{case A} = \frac{\gamma_t}{12} (6h\Psi_2 + a_2 k \Psi_4)$$

$$\text{case B} = \frac{\gamma_t}{12} (6f\Psi_2 + a_2 k_1 \Psi_4)$$

$$I_o = \int_o^{L_2} \mathfrak{M}_o ds = \gamma'_t a_2 k \left[h s_o \Psi_2 + a_2 \left(s_o + \frac{h}{2} \right) \Psi_4 + \frac{2}{3} a_2^2 \Psi_6 \right]$$

$$I_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} ds = 1.07 \gamma_m s_o \Psi_2$$

$$I'_m = \int_o^{L_2} y_2 ds = a_2 \Psi_2$$

$$I'_h = \int_o^{L_2} y_2^2 ds = a_2^2 \Psi_4$$

$$I'_v = \int_o^{L_2} \mathfrak{M}_v y_2 ds =$$

$$\text{case A} = \frac{\gamma_t}{12} a_2 (6h\Psi_4 + a_2 k \Psi_6)$$

$$\text{case B} = \frac{\gamma_t}{12} a_2 (6f\Psi_4 + a_2 k_1 \Psi_6)$$

$$I'_o = \int_o^{L_2} \mathfrak{M}_o y_2 ds = \gamma'_t a_2^2 k \left[h s_o \Psi_4 + a_2 \left(s_o + \frac{h}{2} \right) \Psi_6 + \frac{2}{3} a_2^2 \Psi_8 \right]$$

$$I'_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} y_2 ds = 1.07 \gamma_m s_o a_2 \Psi_4$$

in which :

$$\Psi_o = \int_o^{L_2} \sqrt{1 + 4a_2^2 x^2}\, dx$$

$$\Psi_2 = \int_o^{L_2} x^2 \sqrt{1 + 4a_2^2 x^2}\, dx$$

. . . . . .

. . . . . .

$$\Psi_8 = \int_o^{L_2} x^8 \sqrt{1 + 4a_2^2 x^2}\, dx$$

It will again be noted that all the integrals I′ are divisible by the factor a_2.

As has been done for the preceding cases, we give the formulae for solving the calculation for M and H.

Case A :

$$
\left.
\begin{aligned}
M_v &= \frac{\gamma_t}{12} a_2 k \frac{(\Psi_4)^2 - \Psi_2\Psi_6}{(\Psi_2)^2 - \Psi_0\Psi_4} \\
H_v &= \frac{\gamma_t}{12}\left[\frac{6h}{a_2} - k\frac{\Psi_0\Psi_6 - \Psi_2\Psi_4}{(\Psi_2)^2 - \Psi_0\Psi_4}\right] \\
M_o &= \gamma'_t a_2{}^2 k\left[\left(s_o + \frac{h}{2}\right)\frac{(\Psi_4)^2 - \Psi_2\Psi_6}{(\Psi_2)^2 - \Psi_0\Psi_4} + \frac{2}{3}a_2\frac{\Psi_4\Psi_6 - \Psi_2\Psi_8}{(\Psi_2)^2 - \Psi_0\Psi_4}\right] \\
H_o &= \gamma'_t k\left[s_o h - a_2\left(s_o + \frac{h}{2}\right)\frac{\Psi_0\Psi_6 - \Psi_2\Psi_4}{(\Psi_2)^2 - \Psi_0\Psi_4} - \frac{2}{3}a_2{}^2\frac{\Psi_0\Psi_8 - \Psi_2\Psi_6}{(\Psi_2)^2 - \Psi_0\Psi_4}\right] \\
M_{pp} &= 0 \\
H_{pp} &= 1.07\frac{\gamma_m s_o}{a_2}
\end{aligned}
\right\} \quad (21_A)
$$

Case B :

$$
\left.
\begin{aligned}
M_v &= \frac{\gamma_t}{12} a_2 k \frac{(\Psi_4)^2 - \Psi_2\Psi_6}{(\Psi_2)^2 - \Psi_0\Psi_4} \\
H_v &= \frac{\gamma_t}{12}\left[\frac{6f}{a_2} - k_1\frac{\Psi_0\Psi_6 - \Psi_2\Psi_4}{(\Psi_2)^2 - \Psi_0\Psi_4}\right] \\
M_o &= \gamma'_t a_2{}^2 k\left[\left(s_o + \frac{h}{2}\right)\frac{(\Psi_4)^2 - \Psi_2\Psi_6}{(\Psi_2)^2 - \Psi_0\Psi_4} + \frac{2}{3}a_2\frac{\Psi_4\Psi_6 - \Psi_2\Psi_8}{(\Psi_2)^2 - \Psi_0\Psi_4}\right] \\
H_o &= \gamma' k\left[s_o h - a_2\left(s_o + \frac{h}{2}\right)\frac{\Psi_0\Psi_6 - \Psi_2\Psi_4}{(\Psi_2)^2 - \Psi_0\Psi_4} - \frac{2}{3}a_2{}^2\frac{\Psi_0\Psi_8 - \Psi_2\Psi_6}{(\Psi_2)^2 - \Psi_0\Psi_4}\right] \\
M_{pp} &= 0 \\
H_{pp} &= 1.07\frac{\gamma_m s_o}{a_2}
\end{aligned}
\right\} \quad (21_B)
$$

The integrals Ψ are easily dealt with by making them rational by putting :

$$\sqrt{1 + 4a_2{}^2x^2} = 2a_2x + t$$

Writing in addition :

$$\alpha = 2\,\frac{H_2}{L_2}\;;\; n = -\alpha + \sqrt{1+\alpha^2}$$

Substituting, an easy but laborious matter, we get :

$$\Psi'_0 = \frac{L_2}{4\alpha}\left[-\frac{n^2}{2} - 2\log_e n + \frac{1}{2n^2}\right] = \frac{L_2}{4\,\alpha}\,A_0$$

$$\Psi'_2 = \frac{{L_2}^3}{64\alpha^3}\left[-\frac{n^2}{4} + 2\log_e n + \frac{1}{4n^2}\right] = \frac{{L_2}^3}{64\alpha^3}\,A_2$$

$$\Psi'_4 = \frac{{L_2}^5}{1024\alpha^5}\left[-\frac{n^6}{6} + \frac{n^4}{2} + \frac{n^2}{2} - 4\log_e n - \frac{1}{2n^2} - \frac{1}{2n^4} + \frac{1}{6n^6}\right] = \frac{{L_2}^5}{1024\alpha^5}\,A_4$$

$$\Psi'_6 = \frac{{L_2}^7}{16384\alpha^7}\left[-\frac{n^8}{8} + \frac{2n^6}{3} - n^4 - 2n^2 + 10\log_e n + \frac{2}{n^2} - \frac{1}{n^4} - \frac{2}{3n^6} + \frac{1}{8n^8}\right] = \frac{{L_2}^7}{16384\alpha^7}\,A_6$$

$$\Psi'_8 = \frac{{L_2}^9}{262144\alpha^9}\left[-\frac{n^{10}}{10} + \frac{3n^8}{4} - \frac{13n^6}{6} + 2n^4 + 7n^2 - 28\log_e n - \frac{7}{n^2} - \frac{2}{n^4} + \frac{13}{6n^6} - \frac{3}{4n^8} + \frac{1}{10n^{10}}\right] = \frac{{L_2}^3}{262144\alpha^9}\,A_8$$

And again :

$$\frac{1}{(4\alpha)^4}\cdot\frac{(A_4)^2 - A_2A_6}{(A_2)^2 - A_0A_4} = I$$

$$\frac{1}{(4\alpha)^2}\cdot\frac{A_0A_6 - A_2A_4}{(A_2^2) - A_0A_4} = II$$

$$\frac{1}{(4\alpha)^6}\cdot\frac{A_4A_6 - A_2A_8}{(A_2)^2 - A_0A_4} = III$$

$$\frac{1}{(4\alpha)^4}\cdot\frac{A_0A_8 - A_2A_6}{(A_2)^2 - A_0A_4} = IV$$

by making the necessary calculations, we get :

Case A :

$$\left.\begin{aligned} M_v &= \frac{\gamma_t}{12} k H_2 L_2^2 \text{ I} \\ H_v &= \frac{\gamma_t}{12} \frac{L_2^2}{H_2} (6h - k H_2 \text{ II}) \\ M_o &= \gamma'_t k H_2^2 \left[\left(s_o + \frac{h}{2}\right) \text{I} + \frac{2}{3} H_2 \text{ III}\right] \\ H_o &= \gamma'_t k \left[h s_o - H_2 \left(s_o + \frac{h}{2}\right) \text{II} - \frac{2}{3} H_2 \text{ IV}\right] \\ M_{pp} &= 0 \\ H_{pp} &= 1.07 \frac{\gamma_m s_o}{a_2} \end{aligned}\right\} \quad (21_A)'$$

Case B :

$$\left.\begin{aligned} M_v &= \frac{\gamma_t}{12} k_1 H_2 L_2^2 \text{ I} \\ H_v &= \frac{\gamma_t}{12} \frac{L_2^2}{H_2} (6f - k_1 H_2 \text{ II}) \\ M_o &= \gamma'_t k H_2^2 \left[\left(s_o + \frac{h}{2}\right) \text{I} + \frac{2}{3} H_2 \text{ III}\right] \\ H_o &= \gamma'_t k \left[h s_o - H_2 \left(s_o + \frac{h}{2}\right) \text{II} - \frac{2}{3} H_2 \text{ IV}\right] \\ M_{pp} &= 0 \\ H_{pp} &= 1.07 \frac{\gamma_m s_o}{a_2} \end{aligned}\right\} \quad (21_B)'$$

The quantities I, II, III and IV are functions of the only variable α, i. e. of the ratio between the rise and the chord of the arch and can be calculated by developing the expressions found beforehand.

The calculations are most laborious and the results are given in the ensuing tables.

We have taken values of α varying by steps of 0.10 from $\alpha_1 = 1.50$ to $\alpha_4 = 2.80$.

α	1.50	1.60	1.70	1.80	1.90	2.00	2.10
I	0.01525	0.01542	0.01558	0.01574	0.01590	0.01605	0.01622
II	0.1770	0.1780	0.1788	0.1796	0.1805	0.1813	0.1818
III	0.002711	0.002741	0.002780	0.002820	0.002860	0.002900	0.002939
IV	0.03615	0.03618	0.03621	0.03624	0.03628	0.03632	0.03636

α	2.20	2.30	2.40	2.50	2.60	2.70	2.80
I	0.01638	0.01655	0.01672	0.01689	0.01705	0.01719	0.01733
II	0.1823	0.1827	0.1831	0.1834	0.1836	0.1837	0.1838
III	0.002978	0.003017	0.003056	0.003095	0.003134	0.003173	0.003212
IV	0.03641	0.03645	0.03649	0.03654	0.03659	0.03663	0.03666

b) Arch with elliptic axial curve.

The equation of the axial curve can still be obtained from (19) :

$$y_2 = L_2 (1 - e^2)^{\pm \frac{1}{2}} \left(1 - \sqrt{1 - \frac{x^2}{L_2^2}}\right) \quad \ldots\ldots\ldots \quad (19)$$

whence :

$$\frac{dy}{dx} = \frac{1}{L_2} (1 - e^2)^{\pm \frac{1}{2}} x \cdot \left(1 - \frac{x^2}{L_2^2}\right)^{-\frac{1}{2}}$$

and

$$ds = \sqrt{1 + \left(\frac{dy}{dx}\right)^2}\, dx = \sqrt{\frac{L_2^2 - \varepsilon^2 x^2}{L_2^2 - x^2}}\, dx$$

taking : $\varepsilon^2 = 1 - (1 - e^2)^{\pm 1}$

The integrals I and I′ are then :

$$I_m = \int_o^{L_2} ds = \int_o^{L_2} \sqrt{\frac{L_2^2 - \varepsilon^2 x^2}{L_2^2 - x^2}}\, dx = \Omega_o$$

(this integral represents the evolution of a quarter of ellipse) :

$$I_h = \int_o^{L_2} y_2 ds = L_2\ (1 - e^2)^{\pm\frac{1}{2}} \left(\Omega_o - \frac{\Omega'_o}{L_2}\right)$$

$$I_v = \int_o^{L_2} \mathfrak{M}_v ds =$$

$$\text{case A} = \frac{\gamma_t}{12}\ (6h\Omega_2 + a_2 k \Omega_4)$$

$$\text{case B} = \frac{\gamma_t}{12}\ (6f\Omega_2 + a_2 k_1 \Omega_4)$$

$$I_o = \int_o^{L_2} \mathfrak{M}_o ds = \gamma'_t k a_2 \left[h s_o \Omega_2 + a_2 \left(s_o + \frac{h}{2}\right) \Omega_4 + \frac{2}{3} a_2^2 \Omega_6\right]$$

$$I_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} ds = 1.07 \gamma_m s_o \Omega_2$$

$$I'_m = \int_o^{L_2} y_2 ds = L_2 (1 - e^2)^{\pm\frac{1}{2}} \left(\Omega_o - \frac{\Omega'_o}{L_2}\right)$$

$$I'_h = \int_o^{L_2} y_2^2 ds = L_2^2 (1 - e^2)^{\pm\frac{1}{2}} \left[2\left(\Omega_o - \frac{\Omega'_o}{L_2}\right) - \frac{\Omega_2}{L_2}\right]$$

$$I'_v = \int_o^{L_2} \mathfrak{M}_v y_2 ds =$$

$$\text{case A} = \frac{\gamma_t}{12} L_2 (1 - e^2)^{\pm\frac{1}{2}} \left[6h\left(\Omega_2 - \frac{\Omega'_2}{L_2}\right) + a_2 k \left(\Omega_4 - \frac{\Omega_4}{L_2}\right)\right]$$

$$\text{case B} = \frac{\gamma_t}{12} L_2 (1 - e^2)^{\pm\frac{1}{2}} \left[6f\left(\Omega_2 - \frac{\Omega'_2}{L_2}\right) + a_2 k_1 \left(\Omega_4 - \frac{\Omega'_4}{L_2}\right)\right]$$

$$I_o' = \int_o^{L_2} \mathfrak{M}_o y_2 ds = k\Upsilon'_t L_2 a_2 (1 - e^2)^{\pm\frac{1}{2}} \left[h s_o \left(\Omega_2 - \frac{\Omega'^2}{L_2} \right) + \right.$$

$$\left. + a_2 \left(s_o + \frac{h}{2} \right) \left(\Omega_4 - \frac{\Omega'^4}{L_2} \right) + \frac{2}{3} a_2^2 \left(\Omega_6 - \frac{\Omega'^6}{L_2} \right) \right]$$

$$I'_{pp} = \int_o^{L_2} \mathfrak{M}_{pp} y_2 ds = 1.07 \Upsilon_m s_o \left[\Omega_2 - \frac{\Omega'^2}{L_2} \right]$$

if we write :

$$\Omega_o = \int_o^{L_2} \sqrt{\frac{L_2^2 - \varepsilon^2 x^2}{L_2^2 - x^2}}\, dx \qquad \Omega'_o = \int_o^{L_2} \sqrt{L_2^2 - \varepsilon^2 x^2}\, dx$$

$$\Omega_2 = \int_o^{L_2} x^2 \sqrt{\frac{L_2^2 - \varepsilon^2 x^2}{L_2^2 - x^2}}\, dx \qquad \Omega'_2 = \int_o^{L_2} x^2 \sqrt{L_2^2 - \varepsilon^2 x^2}\, dx$$

. . . . . .

$$\Omega_6 = \int_o^{L_2} x^6 \sqrt{\frac{L_2^2 - \varepsilon^2 x^2}{L_2^2 - x^2}}\, dx \qquad \Omega'_6 = \int_o^{L_2} x^6 \sqrt{L_2^2 - \varepsilon^2 x^2}\, dx$$

The integrals Ω and Ω' now have to be calculated. To calculate the integrals Ω which fall into the elliptical integral class, it will be noted that :

$$\sqrt{\frac{L_2^2 - \varepsilon^2 x^2}{L_2^2 - x^2}} = \sqrt{\frac{1 - \dfrac{\varepsilon^2}{L_2^2} x^2}{1 - \dfrac{x^2}{L_2^2}}}$$

and putting :

$$\frac{x^2}{L_2^2} = \sin^2 \varphi$$

whence :

$$dx = L_2 \cos\varphi d\varphi$$

and

$$\sqrt{\frac{1 - \dfrac{\varepsilon^2}{L_2^2} x^2}{1 - \dfrac{x^2}{L_2^2}}}\, dx = L_2 \sqrt{1 - \varepsilon^2 \sin^2 \varphi}\, d\varphi$$

and consequently :

$$\Omega_0 = L_2 \int_0^{\frac{\pi}{2}} \sqrt{1 - \varepsilon^2 \sin^2\varphi}\, d\varphi$$

$$\Omega_2 = L_2^3 \int_0^{\frac{\pi}{2}} \sin^2\varphi \sqrt{1 - \varepsilon^2 \sin^2\varphi}\, d\varphi$$

$$\Omega_4 = L_2^5 \int_0^{\frac{\pi}{2}} \sin^4\varphi \sqrt{1 - \varepsilon^2 \sin^2\varphi}\, d\varphi$$

$$\Omega_6 = L_2^7 \int_0^{\frac{\pi}{2}} \sin^6\varphi \sqrt{1 - \varepsilon^2 \sin^2\varphi}\, d\varphi$$

By developing the radical by expanding the square root term, which appears in all the integrals according to the binomial formula, restricting oneself to the first three terms, which is more than enough, and in making the easy calculations, we get :

$$\Omega_0 = \frac{\pi}{2} L_2 \left(1 - \frac{\varepsilon^2}{4} - \frac{3\varepsilon^4}{64}\right) = 1.5708\, L_2\, B_0$$

$$\Omega_2 = \frac{\pi}{4} L_2^3 \left(1 - \frac{3\varepsilon^2}{8} - \frac{5\varepsilon^4}{64}\right) = 0.7854\, L_2^3\, B_2$$

$$\Omega_4 = \frac{3\pi}{16} L_2^5 \left(1 - \frac{5\varepsilon^2}{12} - \frac{35\varepsilon^4}{384}\right) = 0.5890\, L_2^5\, B_4$$

$$\Omega_6 = \frac{5\pi}{32} L_2^7 \left(1 - \frac{7\varepsilon^2}{64} - \frac{63\varepsilon^4}{640}\right) = 0.4909\, L_2^7\, B_6$$

indicating by B_0, B_2, B_4, B_6, the functions of ε shown in brakets.

The integrals Ω' can be calculated as follows :

Since :

$$\sqrt{L_2^2 - \varepsilon^2 x^2} = L_2 \sqrt{1 - \frac{\varepsilon^2}{L_2^2} x^2}$$

we put : $\frac{\varepsilon^2}{L_2^2} x^2 = \sin^2 \varphi$ (this being permissible, given that $\frac{\varepsilon^2}{L_2^2} x^2 < 1$)

we then have $dx = \frac{L_2}{\varepsilon} \cos\varphi \; d\varphi$

whence :

$$\Omega'_o = \frac{{L_2}^2}{\varepsilon} \int_o^{\frac{\pi}{2}} \cos^2\varphi \; d\varphi$$

$$\Omega'_2 = \frac{{L_2}^4}{\varepsilon^3} \int_o^{\frac{\pi}{2}} \sin^2\varphi \; \cos^2\varphi \; d\varphi$$

$$\Omega'_4 = \frac{{L_2}^6}{\varepsilon^5} \int_o^{\frac{\pi}{2}} \sin^4\varphi \; \cos^2\varphi \; d\varphi$$

$$\Omega'_6 = \frac{{L_2}^8}{\varepsilon^7} \int_o^{\frac{\pi}{2}} \sin^6\varphi \; \cos^2\varphi \; d\varphi$$

By easy substitutions we then get :

$$\Omega'_0 = \frac{{L_2}^2}{2\varepsilon} \left(\text{arc. sin}\varepsilon + \varepsilon\sqrt{1 - \varepsilon^2} \right) = \frac{{L_2}^2}{2} B'_0$$

$$\Omega'_2 = \frac{{L_2}^4}{8\varepsilon^3} \left(\text{arc. sin}\varepsilon - \varepsilon\sqrt{1 - \varepsilon^2} + 2\varepsilon^3\sqrt{1 - \varepsilon^2} \right) = \frac{{L_2}^4}{8\varepsilon^2} B'_2$$

$$\Omega'_4 = \frac{{L_2}^6}{48\varepsilon^5} \left(3 \text{ arc. sin}\varepsilon - 3\varepsilon\sqrt{1 - \varepsilon^2} - 2 \; \varepsilon^3\sqrt{1 - \varepsilon^2} + 8\varepsilon^5\sqrt{1 - \varepsilon^2} \right) = \frac{{L_2}^6}{48\varepsilon^4} B'_4$$

$$\Omega'_6 = \frac{{L_2}^8}{384\varepsilon^7} \left(15 \text{ arc. sin}\varepsilon - 15 \; \varepsilon\sqrt{1 - \varepsilon^2} - 10 \; \varepsilon^3\sqrt{1 - \varepsilon^2} - 8\varepsilon^5\sqrt{1 - \varepsilon^2} + 48\varepsilon^7\sqrt{1 - \varepsilon^2} \right) = \frac{{L_2}^8}{384\varepsilon^6} B'_6$$

the sense of the symbols B'_0, B'_2, B'_4 and B'_6 being obvious.

In the present case, there is no need to determine the formulae for final solutions giving the values for M and H; it would appear to be simpler and more suitable to determine the values of integrals I and I' from where one can then get the parameters mentioned above.

We would point out that the values given for the various integrals I are all divisible by L_2 and the integrals I′ by $L_2^2 (1 - e^2)^{\pm \frac{1}{2}}$ which, as can easily be seen, will not affect the values of M and H.

We then get :

Case A :

$$
\left.
\begin{aligned}
{}_m &= \frac{\pi}{2} B_0 \\
{}_h &= \frac{L_2}{2} (1 - e^2)^{\pm \frac{1}{2}} \ (\pi B_0 - B'_0) \\
{}_v &= \frac{\pi}{8} \gamma_t L_2^2 \left(h B_2 + \frac{1}{8} H_2 k B_4 \right) \\
{}_o &= \frac{\pi}{4} \gamma'_t k H_2 \left[h s_o B_2 + \frac{3}{4} H_2 \left(s_0 + \frac{h}{2} \right) B_4 + \frac{5}{12} H_2^2 B_6 \right] \\
{}_{pp} &= 1.07 \frac{\pi}{4} \gamma_m s_o L_2^2 B_2 \\
{}_m &= \frac{1}{2} (\pi B_0 - B'_0) \\
{}_h &= L_2 (1 - e^2)^{\pm \frac{1}{2}} \left[\pi \left(B_0 - \frac{B_2}{4} \right) - B'_0 \right] \\
{}_v &= \frac{\gamma_t}{12} L_2^2 \left[6h \left(\frac{\pi}{4} B_2 - \frac{B'_2}{8\varepsilon^2} \right) + k H_2 \left(\frac{3}{16} \pi B_4 - \frac{B'_4}{48\varepsilon^4} \right) \right] \\
{}_o &= \gamma'_t k H_2 \left[h s_o \left(\frac{\pi}{4} B_2 - \frac{B'_2}{8\varepsilon^2} \right) + H_2 \left(s_o + \frac{h}{2} \right) \left(\frac{3}{16} \pi B_4 - \frac{B'_4}{48\varepsilon^4} \right) \right. \\
&\qquad \left. + H_2^2 \left(\frac{5}{48} \pi B_6 - \frac{B'_6}{576\varepsilon^6} \right) \right] \\
{}_{pp} &= 1.07 \gamma_m s_o L_2^2 \left[\frac{\pi}{4} B_2 - \frac{B'_2}{8\varepsilon^2} \right]
\end{aligned}
\right\} \qquad (22_A)
$$

Case B :

$$I_m = \frac{\pi}{2} B_0$$

$$I_h = \frac{L_2}{2}(1 - e^2)^{\pm\frac{1}{2}} (\pi B_0 - B'_0)$$

$$I_y = \frac{\pi}{8} \gamma_t L_2^2 \left(f B_2 + \frac{1}{8} H_2 k_1 B_4 \right)$$

$$I_o = \frac{\pi}{4} \gamma_t k H_2 \left[h s_o B_2 + \frac{3}{4} H_2 \left(s_o + \frac{h}{2} \right) B_4 + \frac{5}{12} H_2^2 B_6 \right]$$

$$I_{pp} = 1.07 \frac{\pi}{4} \gamma_m s_o L_2^2 B_2$$

$$I'_m = \frac{1}{2}(\pi B_0 - B'_0) \qquad (22_B)$$

$$I'_h = L_2(1 - e^2)^{\pm\frac{1}{2}} \left[\pi \left(B_0 - \frac{B_2}{4} \right) - B'_0 \right]$$

$$I'_v = \frac{\gamma_t}{12} L_2^2 \left[6f \left(\frac{\pi}{4} B_2 - \frac{B'_2}{8\varepsilon^2} \right) + k_1 H_2 \left(\frac{3\pi}{16} B_4 - \frac{B'_4}{48\varepsilon^4} \right) \right]$$

$$I'_o = \gamma'_t k H_2 \left[h s_t \left(\frac{\pi}{4} B_2 - \frac{B'_2}{8\varepsilon^2} \right) + H_2 \left(s_o + \frac{h}{2} \right) \left(\frac{3\pi}{16} B_4 - \frac{B'_4}{48\varepsilon^4} \right) + \right.$$

$$\left. + H_2^2 \left(\frac{5\pi}{48} B_6 - \frac{B'_6}{576\varepsilon^6} \right) \right]$$

$$I'_{pp} = 1.07 \gamma_m s_o L_2^2 \left[\frac{\pi}{4} B_2 - \frac{B'_2}{8\varepsilon^2} \right]$$

It did not seem desirable to give a table of numerical values of B and B′, as functions of ε, on account of the large number of values of ε which would have to be considered.

c) Arch with circular axial curve.

In this case the expressions for I and I′ can be taken directly from equations (22_A) and (22_B) by making :

$$e = \varepsilon = 0$$

In that case, we get :

$$B_0 = B_2 = B_4 = B_6 = 1$$

$$B'_0 = \qquad = 2$$

$$\frac{B'_2}{8\varepsilon^2} = \qquad = \frac{1}{3}$$

$$\frac{B'_4}{48\varepsilon^4} = \qquad = \frac{1}{5}$$

$$\frac{B'_6}{576\varepsilon^6} = \qquad = \frac{2}{21}$$

It is then easy to calculate the values of M and of H, which we give hereafter :

Case A :

$$\left.\begin{aligned}
M_v &= \frac{\gamma_t}{100} L_2^2(4.6427h + 0.4356kH_2) \\
H_v &= \gamma_t L_2(0.5601h + 0.10kH_2) \\
M_o &= \frac{\gamma'_t}{100} kH_2\left[(9.3282hs_o - 5.2203H_2\left(s_o + \frac{h}{2}\right) - 6.7865H_2^2\right] \\
H_o &= \gamma'_t k \frac{H_2}{L_2}\left[1.1202hs_o + 1.1759H_2\left(s_o + \frac{h}{2}\right) + 0.7599H_2^2\right] \\
M_{pp} &= 0.09949\gamma_m s_o L_2^2 \\
H_{pp} &= 1.1985\gamma_m s_o L_2
\end{aligned}\right\} \quad (23_A)$$

Case B :

$$
\left.
\begin{aligned}
M_v &= \frac{\gamma_t}{100} L_2^2 \ (4.6427 f + 0.4356 k_1 H_2) \\
H_v &= \gamma_t L_2 \ (0.5601 f + 0.10 k_1 H_2) \\
M_o &= \frac{\gamma'_t}{100} \ k H_2 \left[9.3282 h s_o - 5.2203 H_2 \left(s_o + \frac{h}{2} \right) - 6.7865 H_2^2 \right] \\
H_o &= \gamma'_t k \ \frac{H_2}{L_2} \left[1.1202 h s_o + 1.1759 H_2 \left(s_o + \frac{h}{2} \right) + 0.7599 H_2^2 \right] \\
M_{pp} &= 0.09949 \gamma_m s_o L_2^2 \\
H_{pp} &= 1.1985 \gamma_m s_o L_2
\end{aligned}
\right\} \quad (23_B)
$$

Numerical application.

We shall give a practical application of the formulae arrived at, in particular of the equations (20_B), the case of the elliptic arch of variable thickness.

We assume that we have the lining shown in figure 10, for which :

$s_o = 0.40$

$s_i = 0.52$

$s_m = 0.46$

$\varphi = 20^o$

$\operatorname{tg}^2 \left(45^o - \frac{\varphi}{2} \right) = 0.49$

$\gamma_t = \gamma_m = 2\,000$ kg/mc

$\gamma'_t = 2\,000 \times 0.49 = 980$ kg/mc

$h = 18.00 \left(1 - \frac{L_2^2}{L^2} \right) = 15.90$ m

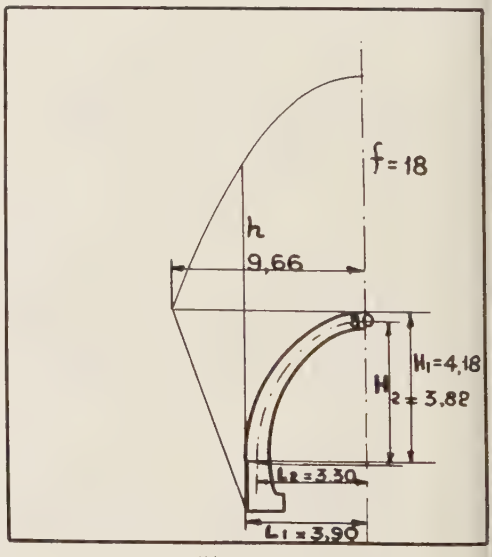

Fig. 10.

$$a_1 = \frac{H_1}{L_1^2} = \frac{4.18}{3.90^2} = 0.274$$

$$a_2 = \frac{H_2}{L_2^2} = \frac{3.82}{3.30^2} = 0.350$$

$$a = \frac{f}{l^2} = \frac{18}{9.66^2} = 0.193$$

$$k = \frac{a_1}{a_2} = 0.783$$

$$k_1 = \frac{a_1 - a}{a_2} = 0.231$$

As on the other hand :

$$L_2 < H_2$$

we have :

$$e^2 = \frac{3.82^2 - 3.30^2}{3.82^2} = 0.2537$$

$$(1 - e^2)^{\frac{1}{2}} = 0.8639$$

We will therefore get :

$$M_v = -\frac{2\,000}{100} \times 3.30^2 (2.5769 \times 18 - 0.4518 \times 0.231 \times 3.82) = -10015.65$$

$$H_v = 2\,000 \times 3.30 (0.6573 \times 18 + 0.0985 \times 0.231 \times 3.82) \times 0.8639 = +67954.00$$

$$M_o = -0.0522 \times 980 \times 0.783 \times 3.82 (15.90 \times 0.40 - 3.82 \times 7.95 - 3.82^2) = +6140.23$$

$$H_o = 980 \times 0.783 \times \frac{3.82}{3.30} (1.31 \times 15.90 \times 0.40 + 1.178 \times 3.82 \times 7.95 + 0.678 \times 3.82^2) \times 0.8639 = +42817.00$$

$$M_{pp} = -0.06145 \times 2\,000 \times 0.46 \times 3.30^2 = -615.65$$

$$H_{pp} = 1.3916 \times 2\,000 \times 0.46 \times 3.30 \times 0.8639 = +3649.00$$

We then find :

$$M_{total} = M_v + M_o + M_{pp} = \quad -4491.07 \text{ kgm}$$

$$H_{total} = H_v + H_o + H_{pp} = \quad +114\,420 \text{ kg}$$

$$\sigma \quad = 18.60 \text{ kg/cm}^2$$

The calculation by the ordinary methods leads (see DESIMON, vol. I, example VI) to the following results :

$$M = -7\,837 \text{ kgm}$$

$$H = 82\,500 \text{ kg}$$

and it follows that : $\sigma = 17.60$ kg/cm^2.

It will be seen that with our formulae, we get a higher pressure, but more centralized, whereas the maxima stresses are fairly concordant.

In other examples of calculation which we have had occasion to develop, we had confirmation of the same results.

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[625 .24 (73)]

Freight cars. — Repair, rebuild or buy new?,

by William Wyer,*

(From the *Railway Age.*)

With the advent of the forty-hour week and the first serious decline in the volume of railway traffic since the war, railroad managements are redoubling their efforts to effect every possible economy in the operation of their properties.

The economics of freight car repairs offers a fertile field for profitable research and analysis, particularly as to the choice between a major repair or rebuilding operation and the purchase of a new car. The subject is particularly timely right now because the 40-hr. week has increased all freight car labor repair costs by 20 %, while for the time being at least freight car builders have not experienced any corresponding increase in costs.

Prior studies of this question have shown consistently that repair costs increase substantially as cars grow older, and conclusions have been reached that the economical point at which an old car should be replaced with a new one may vary between 15 and 30 years, depending upon many factors such as the type of car, its design, its usage, the maintenance policy of the owning carrier, or the proportion of its time on home lines.

While undoubtedly sound, these conclusions are not particularly helpful to the individual railroad facing a specific decision as to whether it should repair or retire an existing car. The application of some general rule-of-thumb based upon averages does not necessarily give it the right answer for its immediate problem. In contrast, what is here presented is a procedure or formula whereby the individual road can make the proper determinations based upon all pertinent factors and feel reasonably sure that it is doing the most economical thing.

An adaptable formula.

The formula is one which can be adapted to the basic maintenance policy of each individual road. Many roads, for example, believe in giving their freight cars general overhaulings at intervals of five to eight years, depending upon the condition of the individual cars at times when they are at home. Others keep their cars going by means of light or heavy running repairs until such time as the entire series seems in need of general rebuilding. This usually occurs sometime between the fifteenth and twentieth year of the car's life and is a substantially more costly job than the periodic general overhaulings.

A determination of the economics of replacing old cars with new would appear to become appropriate at the time of the third general overhaul to an existing freight car, or, if the road contemplates general rebuilding at some point between 15 and 20 years of age, before undertaking such rebuilding.

One side of our equation should reflect the annual cost over the reasonably forseeable future of owning and maintaining the existing car. The other side should show the annual cost over a similar period of owning and maintaining a new car.

Where policy favors the general overhaul method, the information needed to establish the annual cost of owning and maintaining the old car is as follows :

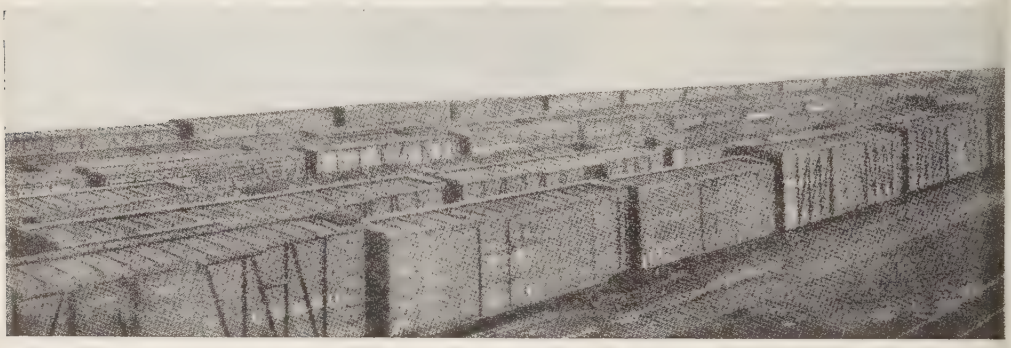
(*) Consulting engineer. Formerly (1943-47) chief executive officer, Central of New Jersey.

1) *Cost of general repairs immediately necessary.* — This should be the result of an actual survey of the car by supervisory officers at the shop where the work would be done. The estimate should include all charges to Account 314, Freight Train Car Repairs, including shop and store expense, together with any charges to capital account immediately or subsequently necessary to keep the car in service over its contemplated future life. This would include such items as AB brakes. Applicable payroll taxes should be added to all labor. Company freight on material used should be added to the cost on an estimated out-of-pocket basis. A percentage should be added to all costs representing the company's experience with respect to personal injuries in freight car repairs or in the mechanical department as a whole.

The description of general repairs set forth above does not include all of the items properly allocable to the cost of repairing freight cars, such as supervision, repairs to shops and power plants, repairs to machinery, stationery and printing, depreciation, etc. In setting up this item in the formula, however, as well as all other similar items, we have tried to follow the principle of including only such costs as will be clearly and directly affected by the decisions resulting from this study. If in any case costs other than those here designated would be affected, it would be not only proper but necessary to take them into consideration.

2) *The probable life before the next succeeding general repair.* — This estimate should be based upon actual experience of the company in question as to the life between the third and fourth general overhaulings on this or similar types of cars.

3) *The annual amount of general repairs* chargeable to the cost of maintaining the existing car in service is (a) the average annual amount of the contemplated expenditure, spread over the probable remaining life on a straight line basis, plus (b) interest at the going equipment trust rate for that company on the undepreciated portion of the cost of general repairs. Over the entire estimated remaining life of the car this undepreciated portion will be one-half of the original repair expenditure.



4) *The annual average running repairs during the remaining period of life.* — This estimate should be based as far as possible on company records, with an appropriate allowance for the increased repair cost ratio on older cars. It should include payroll taxes, personal injuries and company freight.

5) *Estimated annual property taxes* where these can be directly related to the possession of the car. However, where property taxes do not vary with changes

n the owning company's investment, but epend upon net income, net railway oper- ting income, or upon judgment of asses- ors not resulting from the application of ny known precise formula, they should be gnored.

6) *Estimated annual insurance.*

7) *Estimated recoverable value if now crapped.* — In estimating such value all parts should be considered at their realiz- ble scrap value except such as can be sed in repairs to other existing equipment, which should be valued at second-hand prices. Interest on the estimated recover- ble value should be computed at the same ate used in Item 3 above.

8) *Estimated average number of days per year during which the overhauled car would be available for service,* having in nind the probable time it will spend on epair tracks between overhauling and the nd of the period set up in Item 2.

9) *Annual depreciation charges on exist- ng car.* — The daily cost of owning and naintaining the old car would be the sum f Items 3, 4, 5, 6 and 7, divided by Item 8. tem 9 is not utilized at this point in the alculations because it does not involve n actual cash outlay, but is needed later n connection with income tax problems.

Rebuilding costs.

In the case of railroads whose policy avors rebuilding, only minor variations ppear in the information which must be ompiled.

In Item 1, the estimated cost of rebuild- ng should reflect the same charges spe- ified in the case of general overhauling, lus estimated cost of engineering in con- ection with the rebuilding program. If n entire series of cars is to be rebuilt and he amount of work to be done on each ar is substantially identical, regardless of ts individual condition, the cost for the ntire series properly can be estimated as unit. If there is to be a substantial varia- tion in the amount spent on individual cars, however, estimates for each car should be developed as previously described.

In Item 2, the « added life » estimate should reflect the total number of years between rebuilding and retirement. If company policy contemplates a general overhauling within this period, the cost of the overhaul should be included in Item 1.

All the remaining items are identical with those previously indicated in the case of general overhauling, as is the method used in determining the annual and daily cost.

The new car.

The information needed to weight the right-hand scale of our balance — the daily cost of owning and maintaining a new car — should be developed as follows :

1) *Original cost.* — In calculating this cost there should be deducted from the builder's price the purchaser's estimated net profit on any material used in construction of the new car which would move over purchaser's railroad. In computing such net profit, costs of handling should be com- puted on an out-of-pocket basis. Freight on the finished car should be added to the cost to the extent that it appears likely that it will be actually charged, having in mind the possibility of loading the finished, car commercially before it reaches the pur- chaser's rails.

2) *Estimated life of new car* (A).

3) *Estimated life remaining in old car* due for general overhauling or rebuild- ing (B).

4) *Interest on the cost of the new car* should be computed on the basis of 2 % of such cost multiplied by the following factor :

$$\frac{1 + \dfrac{A}{A - B}}{2}$$

The A and B in the factor refer to Items 2 and 3 immediately above. The factor is

necessary to insure that the interest charged against the cost of the new car is computed on the basis of the higher undepreciated cost of the new car during the earliest years of its life — the number of such years being equal to those during which the old car would continue in existence if it were to be repaired or rebuilt.

5) *Depreciation on the new car* based on I. C. C. rates in effect.

6) *Estimated average annual repairs* on the new car for Period B. Where comparison is being made with a car due for general overhauling, Period B will not ordinarily extend to the point where the new car is due for its first general overhauling so that the average annual repairs to the new car would be those expected during the early years of its life, not including any part of the cost of its first general overhauling. Even though the old car might eventually receive another general overhauling after the one immediately contemplated it would not seem to be necessary to extend the comparison beyond the effect of the general overhauling immediately contemplated since any such subsequent general overhauling of the old car would undoubtedly cost substantially more than the first general overhauling of the new car, and the running repairs to both cars would increase in this second cycle as compared with the first cycle under immediate consideration.

When rebuilding is involved, the annual repair charge similarly should not allow for possible rebuilding of the new car after a number of years equivalent to Period B. However, if general overhauling of the new car is contemplated within such a period, the estimated average annual repairs of the new car should be the running repairs up to date of general overhauling plus a proportionate part of the estimated cost of general overhauling applicable to Period B plus the running repairs for that portion of Period B extending beyond the date of general overhauling, all divided by the number of years in Period B. For example, if Period B is estimated to be nine years and it is estimated that the new car would be given a general overhauling in seven years which would carry it through another seven years, the average annual repairs of the new car would be the running repairs during the first seven years plus 2/7 of the estimated cost of general overhauling plus the estimated running repairs of the eighth and ninth years, all divided by nine.

In making the various estimates required in connection with repairs full consideration should be given to features of design, such as multiple-wear wheels, special types of draft gear, type of body construction used and roller bearings, which may affect maintenance costs as compared with those experienced in the past. Repair costs otherwise should be consistent with the repair costs estimated in connection with existing cars, having in mind the type, design and age of the various cars involved.

7) *Property taxes should be computed on the same basis* as suggested in connection with existing cars.

8) *Estimated annual insurance.*

9) *Estimated average number of serviceable days* per year for the new car during Period B.

The daily cost of owning and maintaining the new car would be the sum of Items 4 to 8 above, inclusive, divided by Item 9.

Income tax effect.

After the costs of owning and maintaining old cars and new cars have been determined it probably will be desirable to give consideration to the effect of the purchase of new cars upon the company's income taxes.

The elements of cost as outlined above should be reassembled to list for both the old car and the new car under consideration the average annual deductions for income tax purposes which would be obtainable under the two procedures.

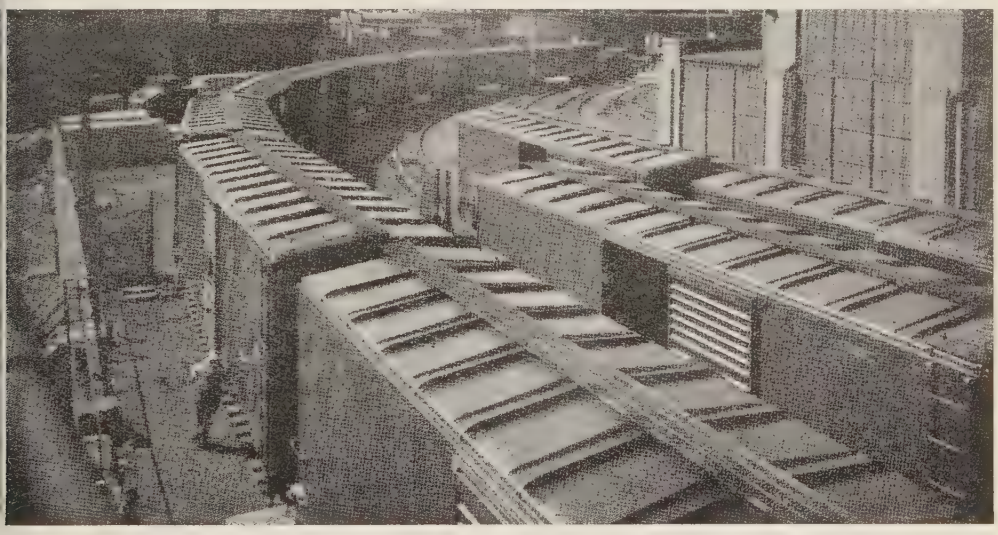
In the case of cars to be rebuilt, if an

of the cost of such rebuilding is chargeable to capital account or if any other capital items are included by reason of being necessary to keep the car in service during its remaining life, these items are of course not deductions for income tax purposes.

It may be found that the deductions for income tax purposes which would be obtained through continued maintenance of the old car are greater than the deductions which would be obtained through the purchase of a new car. In such event the difference represents an increase in taxable income which would be brought about by the purchase of the new car, and if the company expects to be paying income taxes over the period involved, the appropriate income tax percentage should be applied to this increased taxable net income. Conversely, it might be determined that there would be a decrease in income taxes as a result of purchasing a new car.

If a company has carry-forward losses which are available to it for the next two or three years, the income tax effect naturally should not be considered during such period as carry-forward losses are available. Where overhauling is involved, roads in bankruptcy or in weak financial condition can usually get a relatively greater advantage through the purchase of new equipment than a road which is paying income taxes.

If it should become possible to acquire new freight cars on a rental basis, as is now under consideration in some quarters, the economics with respect to income taxes will be substantially changed and it is possible that the acquisition of new cars as compared with maintaining old ones will in practically all instances reduce income taxes.



« Shippers undoubteldy continue to favor lines which can customarily supply them with modern equipment in good repair. »

In order to illustrate how the formula developed above would work, a hypothetical example is given below, based on assumed figures.

Suppose an old car is due for general overhauling. The cost is estimated at $ 1 500, and this is expected to add five

years to the life of the car. It is assumed further that the company can borrow money on equipment-trust certificates at an interest rate of 2 %; that the current net scrap value of the car, allowing second-hand prices for usable units, would be $ 500; that its average running repairs for the next five years after general overhauling would be $ 125; that the annual taxes and insurance on the car would be $ 10 and $ 2 respectively; that 350 serviceable days per year can be expected after overhauling, and that the annual depreciation on the car as it stands on the books would be $ 70.

Table I shows the daily cost of owning and maintaining this car during the next five years.

If a new car were to be purchased to replace this car, let us assume it would cost $ 4 800 and would have an estimated life of 24 years. Further assumptions are that its annual depreciation under I. C. C. regulations, which of course allow for some recoverable scrap value at the end of its life, would be $ 175; that the average running repairs during the first five years of its life would be $ 25; that taxes and insurance would be $ 20 and $ 4, respectively, and that it would provide an annual average of 360 serviceable days for the first five years of its life. The daily cost of owning and maintaining this new car during the first five years is shown in Table II.

Table III shows the deductions for income tax purposes in connection with the two cars. The difference of $ 223 represents the estimated increase in taxable net income as a result of purchasing the new car. If the company expects to be in an income-taxe category during the entire period and its effective rate is 38 %, the increase in income taxes as a result of purchasing the new car would therefore average $ 85, or 24 cents per serviceable car day. These calculations are summarized in Table IV.

TABLE I.

Daily cost of ownership of an old car requiring general overhauling.

Cost of repairs, $1 500; added life, 5 years.

1/5 of cost of general overhauling	$300
Interest on undepreciated cost of general overhauling, 1/2 of 2 % on $1 500	15
Running repairs	125
Taxes	10
Insurance	2
Interest on scrap value at 2 %	10
Total annual cost	$462
Serviceable days	350
Average cost per serviceable day	$1.32

TABLE II.

Daily cost of a new car for the first five years.

Interest, 2 % on $4 800, multiplied by $\frac{1 + 19/24}{2}$	$ 86
Depreciation	175
Repairs	25
Taxes	20
Insurance	4
Total annual cost	$310
Serviceable days	360
Average cost per serviceable day	$0.86

TABLE III.

Comparative expense deductions from annual income affecting income taxes.

	Old car	New car
General overhauling	$300	...
Interest	...	$60
(This is the average annual interest on equipment trusts at 2 % on which a 25 % down payment has been made with installments to mature in 15 years.)		
Running repairs	125	25
Property taxes	10	20
Insurance	2	4
Depreciation	70	125
Total	$507	$284

TABLE IV.

Calculation of income-tax effect.

	Old car	New car	Saving
Taxable increase in net income	...	$223.00	...
Tax at 38 %	...	85.00	...
Tax per serviceable car-day	...	0.24	...
Cost per car-day :			
Including effect on income taxes. . .	$1.32	1.10	$0.22
Excluding effect on income taxes . .	1.32	0.86	0.46

The break-even point.

From the administrative point of view he problem of the individual railroad is ot squarely met by the above example, ince no two cars of any one series will ost exactly the same for general overhaul- ng. As the cars are inspected upon reach- ng back-shop repair yards it will be found hat the cost of repairs covers a wide range. It may very well be that some of the ars can be repaired economically, while thers clearly should be scrapped.

From the practical point of view, there- ore, the railroad needs to establish a limit p to which it will repair cars and beyond hich it will not. This point, of course, s reached when the average daily cost of wning and maintaining the overhauled ar is the same as that of owning and aintaining the new car.

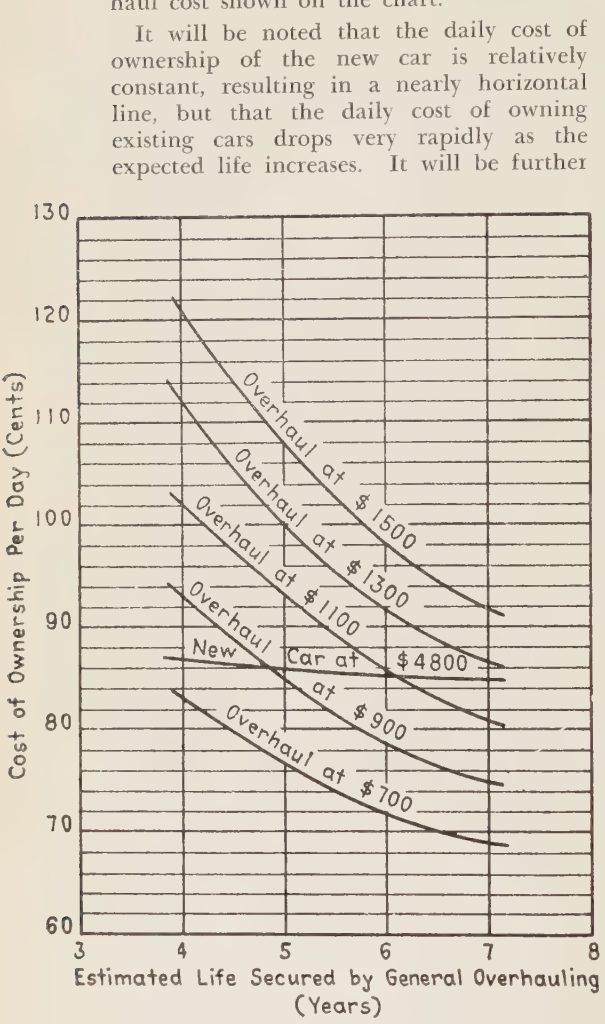
In the case of the set of facts given above, ach variation of $ 100 in the cost of over- auling the existing car makes a difference f about 3.9 cents per day in the saving etween the old and new car costs. This arrived at by deducting $ 20 (or 1/5 of ie $ 100) from the annual cost of over- auling the existing car, and deducting 1 (or 1/2 of 2 % on $ 100) as interest n the undepreciated cost of overhauling. his is a total deduction in the annual ost of maintaining the existing car of $ 21. f the car gives 350 serviceable days this a reduction of exactly 6 cents per day. This also affects the income tax calcula- on through reducing the income deduc- tions in the case of the old car by 1/5 of $ 100, or $ 20. This reduces the increased income tax by reason of purchasing the new car by 38 % of this amount, or $ 7.60, which, allowing for 360 serviceable days a year, amounts to about 2.1 cents per ser- viceable day. The net of these two figures represents 3.9 cents per day change in favor of the existing car as the result of a reduction of $ 100 in the estimated cost of overhauling.

Since the saving per day for the new car against the old on the basis of the example given is 22 cents, including full effect on income taxes, the cost of overhauling the existing car would have to decrease by enough to wipe out the 22 cents saving before the critical limit would be reached. This reduction would be the result of 22 cents divided by 3.9 cents multiplied by $ 100, or approximately $ 565. On the basis of the facts given, therefore, the company would not be justified in spending more than $ 1 500 minus $ 565, or $ 935, for the general overhauling of an existing car of the type involved.

These breaking points vary, of course, depending upon the facts in each com- parison, so that in actual practice there will be an almost endless number of possible combinations. Certain general character- istics, however, apply to all of these com- binations and it may help to understand them to put some typical figures in the form of a chart, based upon the assump- tions already described. In this chart the daily cost of ownership is plotted vertically, while the estimated life secured from the general overhauling is plotted horizontally. The lines on the chart show the variation in the daily cost of ownership depending upon the life estimated to be secured. All lines include the complete effect of varia- tions in income tax, although this is reflected in the cost of ownership of the existing car instead of in the cost of owner- ship of the new car as in the example given above. This is necessary because these sav- ings in income tax vary with the cost of the overhauling and therefore must be

computed separately for each different overhaul cost shown on the chart.

It will be noted that the daily cost of ownership of the new car is relatively constant, resulting in a nearly horizontal line, but that the daily cost of owning existing cars drops very rapidly as the expected life increases. It will be further



How the life extension secured by overhaul expenditures affects the break-even point with the cost of owning new cars.

noted that on the basis of the various assumptions involved an overhaul costing $ 1 500 is definitely uneconomical as compared with the purchase of a new car at $ 4 800 unless a very long life can surely be obtained, the requirement probably being close to 9 or 10 years. An overhaul of $ 1 300 would only be economical if eight years of life could be assured. An overhaul costing $ 1 100 would probably be economical if more than six years of life is assured, with the breaking point for a $ 900 overhaul being somewhere around five years. If four years or more of life could be secured from a $ 700 overhaul this would appear to be clearly economical.

To read the chart in another way, if it is estimated that only about four years of life can be obtained from overhauling a series of cars it would be cheaper to purchase new cars, except in such cases as the overhaul would cost less than $ 800. If five years of life can be reasonably expected overhauls up to about $ 900 would appear to be justifiable. If six years of life can be obtained the breaking point for overhauls would appear to be close to $ 1 100, while if seven years of life can be obtained the limit can be justifiably raised to slightly over $ 1 200.

The chart just discussed, of course, applies to overhauls designed to put from four to seven years of life into a car. Charts covering rebuilding jobs would be of exactly the same type except that the amounts of money involved would run substantially higher, probably between $ 2 000 or $ 3 000, and the years of life to be added would also run greater than in the case of overhauls.

Reliable cost estimates required.

In some cases it may be that a relatively small expenditure, much less than the cost of general overhauling, will permit the existing car to be returned to service for a short period of time, such as 60 or 90 days. If the additional cost of such an expenditure is less than the per diem rate being paid at that time for foreign-line equipment, it may be desirable to make such expenditure, even though the car is to be retired shortly thereafter and even though new equipment is to be ordered to take its place. This, however, is merely a stop-gap expedient justified at time

when surplus cars are not available on line and where the use of foreign-line cars would be necessary at any increased per diem expense.

Many railroads do not have on hand reliable basic data with which to make many of the estimates required for complete application of the formula outlined above. It has been my experience in such instances that the costs of overhauling or rebuilding frequently are underestimated, and that the probable life added by overhauling or rebuilding often is overestimated. This cannot be attributed to any deliberate attempt to mislead but rather to a real lack of information, plus the laudable human trait of optimism. The chart already discussed emphasizes the great importance of securing accurate data as to those items.

An important first step toward more realistic estimating is the preparation of an analysis of Account 314 over a representative period. The analysis should show:

1) Inspection costs, including shop expense.

2) Other direct labor and material including shop and store expense. This should be separated between (a) running repairs, (b) general overhauling, and (c) rebuilding. The latter two items should be separated further according to the class of car.

3) Amounts collected from foreign lines on A. A. R. billing for repairs to foreign cars.

4) Amounts paid to foreign lines on A. A. R. billing for repairs to system cars. This item should be analyzed according to classes of cars and the amounts so obtained compared with the number of car days of each class off line.

5) Casualties.

6) Other.

Item 2a, less Item 3, divided by the number of car-days of home cars on line during the period involved, will give the average cost of running repairs per car-day for system cars at home. This furnishes a good base upon which to estimate the probable running repairs on any specific class of car while at home, after giving consideration to its type, design and age.

Similarly, Item 4 should furnish a good base upon which to estimate the probable running repairs on the car while on foreign lines.

A spot analysis of the factors enumerated above, while providing satisfactory data for current estimates, still falls far short of the ideal. By the time the next appraisal period arrives, repair experience, price levels, and many other factors may have changed. There is much to be said, therefore, for a continuing record of freight car costs along the lines suggested above. In the past the probable increase in accounting expense has been a major stumbling block, but with the increase in the use of modern machine equipment in railroad accounting this objection is rapidly disappearing.

Additionally, such a running record would emphasize the variations in the daily cost of owning and maintaining cars of varying types, sizes and design, and would stress the importance of giving further consideration to the desirability of variable per diem rates, especially for special service cars and possibly based on age. Such data, especially if it led to higher rates for newer cars, would be a powerful factor in bringing about a general improvement of freight car equipment, with resultant benefits to the entire industry.

Intangible benefits.

Intangible benefits from the modernization of equipment, for which no mathematical allowance can be made in our proposed formula, undoubtedly should influence many close decisions as to the wisdom of overhauling or rebuilding.

Shippers undoubtedly tend to favor lines which customarily can supply them with modern up-to-date equipment in good repair. It might even be that the traffic department could give specific assurances

as to certain business which would be obtainable if new cars were available. Many new cars also are designed to reduce damage claims, to offer lower rolling resistance and thus make possible heavier train loads, to permit operation at higher speeds, to be loaded and unloaded more efficiently, to promote safer operation, or to contribute in other ways to greater efficiency of operation. The owning road will only participate in these benefits to the extent that the car remains on its own rails, and to the extent that other cars in its trains permit full utilization of the advantages of the new car, but as each road makes its own contribution to the general improvement of freight car design, the industry as a whole will begins to reap these benefits.

As far as general public relations are concerned, one of the best advertisements that any railroad can have is a new freight car which immediately catches the public eye, whether of travelers by rail or highway or of those living or working along the line.

Another very important intangible element is the question of shops, shop machinery and shop yard tracks, especially the items of maintenance, insurance and taxes. As already noted, there is no allowance in any of the formula items for these costs. Nevertheless a reduction in the amount of overhauling or rebuilding done in freight car shops undoubtedly effects some saving in such costs. The saving would become most substantial and worthy of attempt at measurement through a special study if the adoption of a general policy of less rebuilding and more purchases of new cars should make it possible to close specific shops or shop buildings or to eliminate the use of particular machines.

It will be noted that the unit costs per day of owning new freight cars and of keeping old cars in repair are both substantially lower, based on the examples here used, than the going per diem rate of $ 1.75. This will be true of almost any set of figures which realistically reflect current costs. This is because the costs entering into the calculation of the per diem rate include all of the overhead costs already mentioned which have been excluded from our figures and also include some allowance for existence of surplus equipment. Unless, therefore, the purchase of new cars will involve a railroad in extensive capital expenditures in order to provide repair facilities for those cars or will result in other increases in overhead costs or undue surpluses of those cars, it would seem to be in the interest of every railroad in the country to provide itself with sufficient freight cars to be sure that it is not paying through heavy per diem expense more than it would cost to own an adequate supply of its own.

In this respect each railroad's problem may well be different than its neighbor's and this article does not attempt to set forth a formula for determining whether a railroad's freight car ownership is sufficient for its needs. But it is believed that the figures, in addition to illustrating a method for making sound decisions regarding rebuilding or overhauling, also clearly point the way to economies available to many railroads through an increase in their existing ownership of freight cars.

CORRIGENDUM

Bulletin for January 1951.

On page 51, 2nd line :

Please read : **Malayan Railway Report for the year 1947,** instead of the year *1948* as quoted.

[385. (09 .2]

OBITUARY.

Léon JACOBS,

Ingénieur des Constructions Civiles.
Directeur Général Honoraire de la Société Nationale des Chemins de fer Vicinaux (Belgium).
Former Member of the Permanent Commission of the International Railway Congress Association.

M. Léon Jacobs, former general manager of the « Société Nationale des Chemins de fer Vicinaux de Belgique » and former member of the Permanent Commission of our Association died on the 19th April 1951.

M. Léon Jacobs was born at Laeken on the 30th September 1880. He proved to be a brilliant scholar and entered the University of Louvain where he obtained his civil engineering degree in 1906. He was engaged that same year as temporary assistant engineer by the « Société Nationale des Chemins de fer Vicinaux ».

Starting from the bottom in this way, he was promoted progressively to the highest grade on the light railways. In 1910, he became assistant engineer of the Limbourg provincial service, after which he spent a short time at headquarters in 1911 to return shortly afterwards as chief provincial engineer for Limbourg of which he was particularly fond, having spent his childhood there.

After a few months in captivity during the first world war, he was recalled to headquarters in 1916 as divisional engineer. During the difficult postwar years, M. Jacobs had to solve the many problems arising as a result of taking over the bankrupt subsidiary companies and the straightening out of the finances of the company. During this same epoch, the first railcars were put into service and the electrification of the system begun.

On the 18th July 1928, M. Léon Jacobs was appointed assistant general manager, and in 1933 he succeeded M. Gaufriez as general manager.

Apart from his great technical abilities, M. Jacobs was the inaugurator of various

measures and steps of a social character for the benefit of the staff of the company under his management.

During the 1940-1945 war, he devoted himself entirely to the organisation of the transport of the country according to the circumstances of the times and never failed in his fight against the demands of the occupying powers.

On the 24th September 1947, M. JACOBS retired, deeply regretted by the whole of his staff who held him in the greatest esteem both on account of his ability as manager and his qualities as a man.

The deceased was a grand officer of the Order of Leopold and the Order of the Crown, and the holder of many other decorations.

M. Léon JACOBS was appointed a member of the Permanent Commission of our Association in 1933, and always showed the greatest interest in its activities.

He took part in the Sessions held in Rome in 1922, London in 1925, Madrid in 1930, Cairo in 1933, Paris in 1937 and Lucerne in 1947, as well as the enlarged meeting of the Permanent Commission in Brussels in 1935 and 1939. His great competence in all matters concerning light railways was always much appreciated by his colleagues in Section V, and he was Vice-President of this Section at the Madrid Congress.

At the Cairo Congress, he was a reporter on Question XII dealing with co-ordinating the working of main line and light railways.

Our regretted colleague will long be remembered and we offer his family our sincerest sympathy.

The Executive Committee.

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PUBLISHED UNDER THE SUPERVISION OF

P. GHILAIN,

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(SEPTEMBER 1951)

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JOUFFROY (L. M.). — De la **substitution progressive de la caisse mobile (container) à la caisse fixe dans les transports par chemin de fer.** (15 000 mots & fig.)

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Revue générale des Chemins de fer, mars, p. 127.
CAUVY. — La **reconstruction du viaduc de Lessart** sur la Rance. (3 000 mots & fig.)

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LANDRIN & JOUANEN. — **Evolution du trafic des fruits et légumes** de la vallée du Rhône. (6 000 mots & fig.)

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Mise en service, en Suisse, de deux **voitures sur pneumatiques.** (300 mots & fig.)

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LE HORGNE. — Les **voitures en acier inoxydable** des Chemins de fer Algériens. (3 000 mots & fig.)

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Revue générale des Chemins de fer, avril, p. 164
WALTER & THOUVENOT. — La **téléphonie matique interurbaine appliquée au chemin de fer.** (1 mots & fig.)

1951 **351** .812 .1
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Modifications apportées, depuis l'origine, à la vention du 31 août 1937 relative **au nouveau régim Chemins de fer français.** (20 000 mots.)

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Chemins de fer et trafic routier dans la défense d (2 000 mots.)

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MANDEL (J.). — **Ecoulement de l'eau sous u de palplanches.** Abaque pour la condition de (5 000 mots & fig.)

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PINÇON (A.). — Etude comparative de la métrie des bétons** de ciment, de bitume et d'argile aux numéros de février 1947, mai 1947, août octobre 1948). (7 000 mots & fig.)

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FOUGEA (E.) & CAYLA (M.). — **Nouvea de mise en précontrainte.** (4 500 mots & fig.)

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LANG (W.). — Das **Bahnumrichterproblem i** men der heutigen Bahn- und Landesstromvers (10 000 Wörter & Abb.)

621 .431 .72
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X (R.). — Über die Bemessung der **elektrischen bertragung bei dieselelektrischen Triebfahrzeugen.** Wörter & Abb.)

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IAEFER (H. H.). — **6 000 P. S. Versuchslokoder Pennsylvania RR, U. S. A., mit Ignitronrichtern.** (1 000 Wörter & Abb.)

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ELING (H.). — Bericht über weitere Vorrichtungen **essung von Bewegungen der Fahrleitungen.** (1 200 r & Abb.)

621 .335 (43)
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ADIGAU (A.). — Die **Arbeitsverluste der Regei 50 Hz Einphasenwechselstrom-Lokomotiven** mit nzierten Einphaseninduktionsmotoren (Bauart). (12 000 Wörter & Abb.)

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ANZ (K.). — **Bogengeschwindigkeit** von Schienentrassenfahrzeugen. (5 000 Wörter & Abb.)

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AUENDORF (H.). — Vorschläge zur Weiterentng der **Strassenbahn-Triebwagen.** (6 000 Wörter ɔ.)

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OPP (A.). — Die **Entwässerung von Bahnanlagen.** Wörter & Abb.)

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ÜCKMANN (B.). — **Brückenmesswesen, Brückengungen und Brückenbelastbarkeit.** (10 000 Wörter.)

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GEL (F.). — **Liegedauer der Schienen.** (1 500 r & Abb.)

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CHTERHANDT (J.). — Gedanken zur Weiterg der **Lokomotivbehandlungsanlagen** (Bw.). (1 700 & Abb.)

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CIESIELSKI (H.). — **Montage von schweren Fertigbetonbauteilen.** Ein vorspannungsloses und Stahleinlagen ersparendes Verfahren besonders für Bogenbrücken. (2 500 Wörter & Abb.)

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Modernising a freight yard on the Santa Fe Railways. (1 600 words & fig.)

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Multi-lingual announcements at Harwich. (1 200 words & fig.)

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Diesel operated mobile crane. (600 words & fig.)

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Steam railcars for Egypt. (2 400 words & fig.)

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(600 words & fig.)

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(500 words & fig.)

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DI RAIMONDO (G.). — **The Italian State Rail**
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Container-trailers in South Africa. (600 words &

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Electric locomotive for Estoril Railway. (2 000
& fig.)

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McARD (G. W.). — **Influence of driving whee**
meter on locomotive design. (2 400 words & fig.)

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CAZENAVE (C. R.). — **French Railways**
service in Paris and Suburbs. (2 000 words & fig

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Norwegian rail-spreading and lifting equipment.
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Reconstruction of London Transport rolling
(2 400 words & fig.)

1950 **621** .138
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Ulster Transport Authority workshops. (1 500
& fig.)

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Girder-type silent rail joint. (600 words & fig.)

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385 (061 (73)
y Mechanical Engineer, November, p. 647.
brake Association holds annual meeting. (8 000
& fig.)

385 (061 (73) & 621 .13 (06 (73)
& 621 .33 (06 (73)
y Mechanical Engineer, November, p. 659.
l supervisors discuss locomotive operation. (8 000
& tables.)

385 (061 .4 (73)
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rical sections have new responsibilities. (13 000
& fig.)

621 .138 .5 (73)
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of modernizing locomotives repaid in 3 years.
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625 .28
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SELL (F. E.). — **Changing over from steam to power.** (2 400 words & fig.)

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. & W. passenger cars. (800 words & fig.)

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ro-motive switchers. (1 200 words & fig.)

621 .39
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CHT (W. A.), KERR (C. Jr.) & PUTZ (T. J.). —
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RRICK (T. J.). — **Control system for a gas turbine locomotive.** (3 500 words & fig.)

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LOTT (J. I.). — **The gas turbine at Dunkirk.**
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CRAMER (R. E.) & JENSEN (R. S.). — **Progress reports of investigation of railroad rails : joint bars : and manganese steel casting.** (42 pages & fig.)

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SIESS (C. P.) & NEWMARK (N. M.). — **Moments in two-way concrete floor slabs.** (122 pages).

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GOMEZ-CORDOBES HERNANDEZ (J. J.). — Momento de inercia de la seccion circular hueca para el material **hormigon armado.** (2 000 palabras & fig.)

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ANGULO ALVAREZ (A.). — La **tecnica del hormigon vibrado.** (3 000 palabras & fig.)

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MENDIZABAL (D.). — Un « **record** » **ferroviario.** (Renovación de un trozo de via en Alemania). (800 palabras & fig.)

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VAN VLIET (N.). — **Los Ferrocarriles Holandeses.** (4 000 palabras & fig.)

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Plan de reconstrucción y mejora de servicio en la Red Nacional de los Ferrocarriles Españoles. (4 000 palabras & fig.)

In Italian.

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1950 **62** (01

Alluminio, nº 6, p. 523.

RUFF (W.). — Sopra un possibile perfezionamento dei classici **apparati fotoelasticimetrici.** (5 000 parole & fig.)

1951 **62** (01 & **669** .71

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GATTO (F.). — Sulle **prove di trazione dinamica.** (1 700 parole & fig.)

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135 tonnellate di anticorodal nella nuova stazione di Roma. (1 200 parole & fig.)

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1950 **624** .7 (45) & **691** (45)

Giornale del Genio civile, dicembre, p. 713.

MORANDI (R.). — Il **nuovo ponte di calcestruzzo precompresso** sul fiume Elsa a Canneto. (4 000 parole & fig.)

1950 **624** & **691**

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FRANCIOSI (V.). — Su alcuni problemi connessi con **la precompressione nelle strutture iperstatiche.** (2 500 parole & fig.)

1951 **624** .2 & **691**

Giornale del Genio civile, gennaio, p. 23.

CESTELLI GUIDI (C.). — Considerazioni sulla **sicurezza delle strutture staticamente indeterminate in conglomerato precompresso.** (1 200 parole.)

1951 **62** (01

Giornale del Genio civile, gennaio, p. 25.

FINZI (L.). — Sul **calcolo dei carichi critici** con il principio dei lavori virtuali. (4 000 parole & fig.)

Ingegneria ferroviaria. (Roma.)

1951 **624** .2

Ingegneria ferroviaria, marzo, p. 171.

POLSONI (G.). — **Il diagramma reale dei max menti flettenti ed il diagramma convenzionale di luppo.** (2 500 parole & fig.)

1951 **656**

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RIGHI (R.). — Il **calcolo dei circuiti di binario** tensione al caso in cui si impiegano relè a due elem (3 500 parole & fig.)

1951 **656** .1 (

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VEZZANI (F.). — Il **costo dei trasporti automobil** al 1º marzo 1951. (10 000 parole & tabelle.)

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GIAGNONI (C.). — Alcune considerazioni **problema della captazione di corrente a locom fermo.** (1 500 parole & fig.)

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Ingegneria ferroviaria, marzo, p. 211.

ALTIERI (M.). — Il **laboratorio di elettrotecn fisica tecnica** dell' Istituto Sperimentale delle (3 000 parole & fig.)

In Netherlands.

Spoor- en Tramwegen. (Utrecht.)

1951 **625** .151

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SCHMELLING (J. C. W.). — Een **symmetrisch 1 : 20 NP 46** bij de Nederlandsche Spoorwegen. woorden & fig.)

1951 **65**

Spoor- en Tramwegen, nr 5, 8 Maart, p. 80; nr Maart, p. 89.

VAN DER MEULEN (J. H.). — Juridische as van de ongelijke **concurrentiestrijd van de Amer Spoorwegen.** (4 000 woorden.)

1951 **351** .812

Spoor- en Tramwegen, Nr 7, 5 April, p. 101; 19 April, p. 125; Nr 9, 3 Mei, p. 141; Nr 11, 3 p. 181; Nr 13, 28 Juni, p. 218.

VAN BALLEGOIJEN DE JONG (J. P. A **Organisatie British Railways.** (12 000 woorden

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1951 **656** .25

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DE BLIECK (V. J. M.). — Enkele bijzonder merken van **N. X.- beveiligingstoestellen.** (3 000 den & fig.)

1951 **385** (

Spoor- en Tramwegen, Nr 9, 3 Mei, p. 144.

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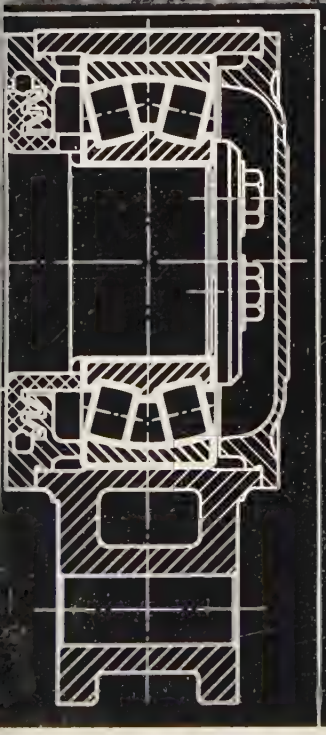

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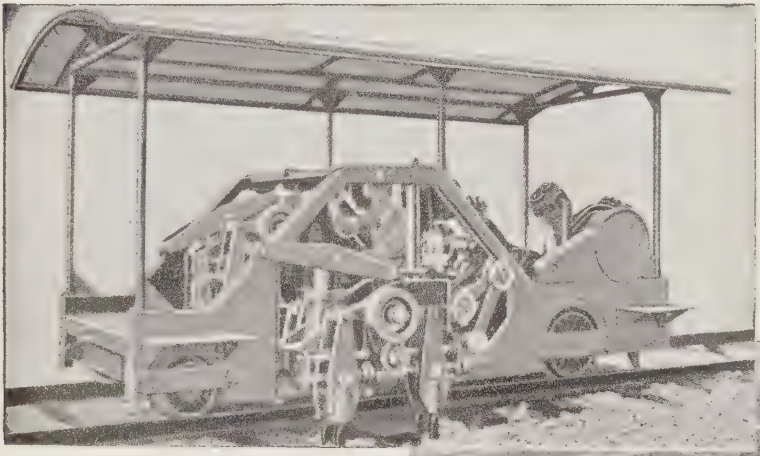

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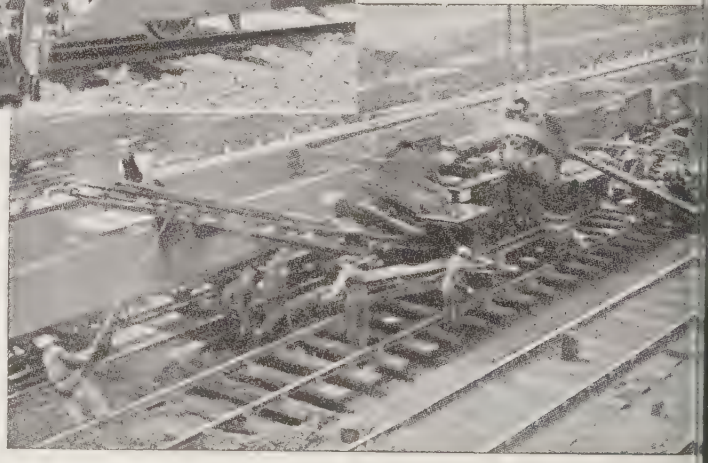

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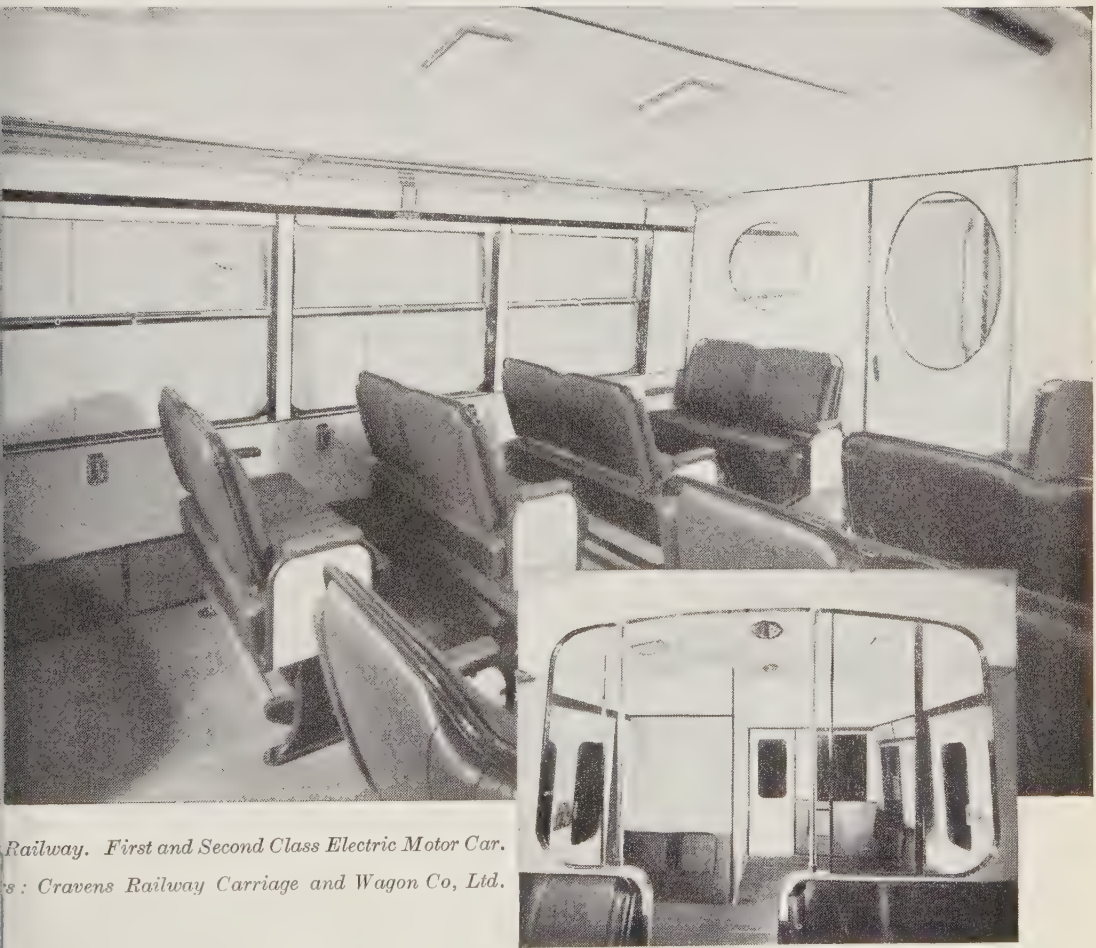

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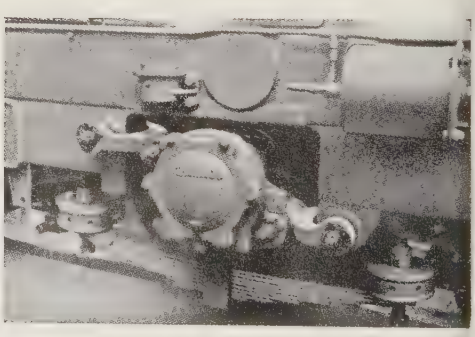

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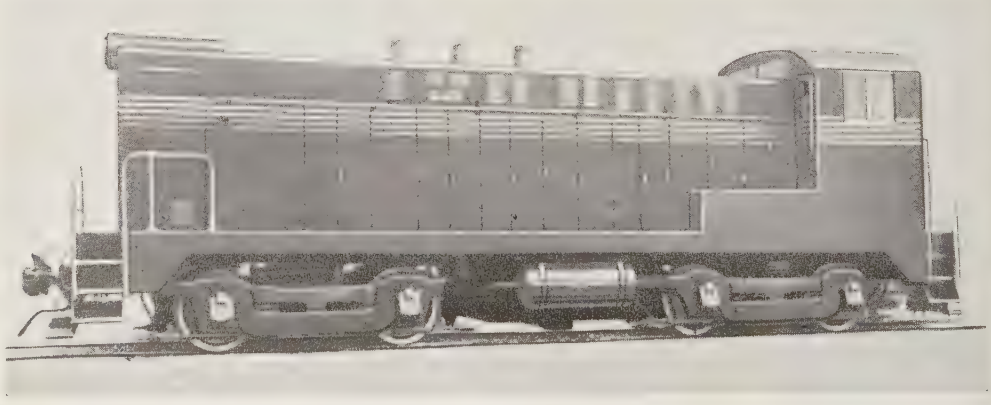

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